Proceedings of the Sixteenth Canadian Soil Mechanics Conference 12-13-14 September 1962: Technical Memorandum No. 82

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OF THE

SIXTEENTH CANADIAN SOIL MECHANICS CONFERENCE

12-13-14 SEPTEMBER 1962

TECHNICAL MEMORANDUM NO. 82

Prepared by

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PREFACE

The Sixteenth Soil Mechanics Conference held in September 1962 at the University of Alberta, under the chairmanship of Dr. R. M. Hardy, was sponsored jointly by the EIC and the Associate Committee on Soil and Snow Mechanics of the National Research Council. Although the conference shared some of the sessions with the Zone "A" Technical Meeting, these proceedings record only the first two days of the three-day conference which were devoted to topics of interest to soil engineers. The proceedings contain summaries of those papers which are to be published by journals; the remaining papers, which are not expected to appear elsewhere in their present form, are printed in their entirety.

Nineteen hundred and sixty-two was the last year in which the ACSSM, through its Subcommittee on Soil Mechanics, was responsible for the organization and sponsorship of the Annual Soil Mechanics Conference. The Seventeenth Conference will be held in 1963 in Ottawa in September, but it will be under the auspices of the EIC.

The need for such a "meeting ground", as these annual conferences have provided for soil mechanics workers in all parts of Canada, was first appreciated in the immediate post-war period by R. F. Legget, Chairman of the Associate Committee on Soil and Snow Mechanics, then on the civil engineering staff of the University of Toronto. The first meeting was held in Ottawa in April 1947 and was attended by all the known Canadian workers in the field at that time, with Dr. L. F. Cooling (of the British Building Research Station) as an honoured guest.

The Division of Building Research was established by the National Research Council later in 1947. Starting in 1948, members of the staff of the Soil Mechanics Section of the Division have carried the administrative responsibility for the organization of the successive conferences, in association with local committees when the meetings were not held in Ottawa, until the Sixteenth Conference of 1962.

At first, these gatherings served the needs of soil mechanics research workers in Canada, providing a useful forum for

discussion. From the outset, the meetings were characterized by the fruitful participation of a few workers from the allied disciplines of pedology and geology, with experts from other branches of applied science often present also, forestry workers having made useful contributions. Mimeographed proceedings were prepared annually by the DBR Soil Mechanics staff not only to provide a useful permanent record but also to make known the results of soil mechanics research for the benefit of engineers engaged in foundation, road and airport design and construction. The conferences gradually attracted the attendance of such engineers and over the years assumed the character of full-fledged professional meetings with as many as 250 engineers and scientists present from all over Canada. With the success of the annual conferences assured, the Associate Committee decided that one of its initial goals had been attained and that the sponsorship of the meetings had really ceased to be a proper function for the National Research Council.

It was agreed that the organizing of such large meetings could best be assumed by an engineering society such as the Engineering Institute of Canada. The Institute welcomed the suggestion and established a Geotechnical Engineering Division of its Committee on Technical Operations to be responsible for future conferences. A gradual transition was arranged, the Fifteenth Conference being sponsored by the NRC and EIC, and the Sixteenth Conference under the auspices of the EIC and NRC. The transfer is now complete, plans for the Seventeenth Conference already being prepared by the Geotechnical Engineering Division of CTO/EIC.

It may be noted that although many of the annual conferences were held in Ottawa, a number were held in various other locations such as Vancouver, Lethbridge, Edmonton, Saskatoon, Winnipeg, Niagara Falls, Montreal, and Halifax, to bring the benefits of such meetings to those who could not travel the long distances involved. Nineteen hundred and sixth-two also marks the last year when the proceedings of the conference will by published by the ACSSM. For this final year, those papers not appearing elsewhere in recognized journals will be published in the proceedings; others will be summarized.

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PROGRAM

of the Engineering Institute of Canada
Zone "A" Technical Conference
in Conjunction with
The Sixteenth Annual Canadian
Soil Mechanics Conference
Edmonton, Alberta, September 12-13-14, 1962

Wednesday, 12 September 1962

Registration

Opening Remarks - Dr. R. M. Hardy, Conference Chairman.
Mr. C. B. Crawford, National Research Council.

TECHNIGAL SESSION #1

"Volume Change Characteristics of Highly Plastic Soils"
C. B. Crawford, Chairman

"Volume Changes in Undisturbed Clay Profiles in Western Canada".
J. J. Hamilton, N. R. C.

"Heave of Spillway Structures on Clay Shales".
R. Peterson and N. Peters, P. F. R. A.

"Practical Experience with Highly Swelling Soil Types".
R. M. Hardy, R. M. Hardy & Assocs., and A. O. Dyregrov,
Underwood, McLellan & Assocs.

Lunch at Athabaska Hall

TECHNICAL SESSION #2

"Physico-Chemical Aspects of Soils"
G. C. McRostie, Chairman

Chairman's Remarks

"Physico-Chemical Phenomena in Soil Materials"
S. Pawluk, University of Alberta (Soil Science Department).

"Measurements of Shear Strength, Plasticity and Water Retention of Clays Related to Interparticle Forces"
B. P. Warkentin, Macdonald College, Montreal.

"Shear Strength of a Remoulded, Normally Consolidated Homionic Clay"
S. Thomson, University of Alberta.

"Engineering Properties of Illite Related to Fabric and Pore Water Composition"
R. M. Quigley, Geocon Ltd.

Business Sessions:

Soil Mechanics Subcommittee of Associate Committee on Soil and Snow Mechanics of the National Research Council.

Geotechnical Division of Committee on Technical Operations of the Engineering Institute of Canada.

Evening Research Seminar

"Research on the Physico-Chemical Aspects of Soils as Related to their Engineering Properties"
Prof. D. L. Townsend, Chairman

Thursday, 13 September 1962

Combined Soil Mechanics Conference and Zone "A" Engineering Institute of Canada

Registration

Opening Remarks: Dr. R.M. Hardy, General Chairman
Mr. F.M. Cazalet, Vice-President, EIC
Prof. J.R. Mantle, Vice-President, EIC
Dean G.W. Govier, University of Alberta

TECHNICAL SESSION #3

Prof. J. Longworth (U. of A.) Chairman

"Surficial Deposits in Alberta"
C.P. Gravenor and L.A. Bayrock, Research Council of Alberta.

"Engineering Aspects of the Great Slave Railway"
J.L. Charles, V.R. Cox and F.L. Peckover.

"Preliminary Soil Mechanics Aspects of the Red River Floodway"
J. Mishtak, Soil Mechanics Water Control & Conservation Branch, Manitoba Department of Agriculture.

Informal Luncheon, Northern Alberta Jubilee Auditorium.

TECHNICAL SESSION #4

C.F. Ripley, Chairman

Chairman's Remarks
"The Columbia River Development -- A Progress Report"
British Columbia Hydro & Power Authority and Consultants.

(a) The Overall Project
(b) The Mica Creek Project
(c) The Arrow Lakes Project
(d) The Duncan Lakes Project

"Foundation Treatment and Construction of Sixteen Miles of Dyke at Manitoba Hydro's Grand Rapids Project"
A. Koropatnick, W.S. Isom and J.R. Rettie, Manitoba Hydro.

Stag Dinner Arranged by Edmonton Branch, E.I.C. at Macdonald Hotel
Speaker: Dr. K.F. Tupper, President, Ewbank, Tupper & Assocs. Ltd. and Past-President, E.I.C.

Friday, September 14

Engineering Institute of Canada
Zone "A" Technical Meeting

TECHNICAL SESSION #5

J. C. Dale, Chairman

Chairman's Remarks

"Design of Pipe Lines to Minimize Brittle Failure"
V. P. Milo, Shell Oil Company.

"The Alberta to California Pipe Line System"
D. P. Smith and/or W. L. Kennedy, Canadian Bechtel Ltd.

"Energy Interchange and a National Grid"
D. Cass-Beggs, Saskatchewan Power Corporation

"The Alberta Electric Power Situation"
G. A. Gaherty, Montreal Engineering Co. Ltd.,

Informal Luncheon Northern Alberta Jubilee Auditorium
Official Opening of New Wing of Engineering Building at the University of Alberta.

TECHNICAL SESSION #6

Dr. R. W. McManus, Chairman

"Oil and Gas Reserves in Western Canada"
J. G. Stabback and R. D. Craig, Alberta Oil and Gas Conservation Board.

"The Remote Control of a Gas Field"
J. H. Schwartz, Timewell Controls Ltd.

Tours of Engineering Laboratories, University Campus.

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SHEAR STRENGTH OF A REMOULDED NORMALLY CONSOLIDATED HOMIONIC CLAY

by

S. Thomson*

SUMMARY

Water in itself and in its behaviour is a complex substance. There are Van der Waals and Coulombic forces operating between the bodies present. Clay particle surfaces and edges, free and adsorbed cations and anions all possessing charges, presumably all interact. Further, charges on the particles are not evenly distributed but may have points of concentration and the cations differ both in size and charge. Thermal energy and the infinite variety of inter-particle relationships would appear to compound an already complex system. In most instances all that can be measured is the net effect of these forces.

This paper is based on the triaxial strength results obtained by compressing a normally consolidated remoulded chemically treated clay soil. It is concluded that both the adsorbed cation complex and the concentration of salts in the pore water influence the shear strength of a soil. The main factor considered to account for the variations in strength are the changes in thickness of the adsorbed film that are brought about by the adsorbed cation or salt concentration in the pore water.

The work that forms the basis of this paper was carried out as a requirement for a Ph. D. degree at the University of Alberta with financial assistance from the Joint Highway Research Program of the Province of Alberta. The author expresses his appreciation of the help and criticism extended to him by many members of the staff. Of particular note are Dr. R. M. Hardy and Professor S. R. Sinclair.

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*See Appendix "A" for affiliation.

The role played by soil physics and physical chemistry in explaining the geotechnical behaviour of clay soils is increasing in importance and is contributing to an undertstanding of this behaviour. This paper presents results obtained from laboratory triaxial tests carried out on a remoulded, normally consolidated clay soil that was modified by having a high proportion of the exchange complex occupied by a single cation species. In addition to the natural soil, the modifications studied were those resulting from the partial saturation of the exchange complex with calcium, magnesium, potassium and sodium cations.

Sample Preparation

The soil used for this program was obtained from a highway cut located on the southern outskirts of Edmonton. The cut was opened in the latter part of the summer of 1959 and the bulk soil sample was dug out of the backslope just above road level in early April of 1960. Vertically, the sample location was between fifteen and twenty feet below the original ground surface. Five hundred pounds of this soil was obtained and allowed to air dry in the laboratory. The material is a glacial lake sediment classified as a highly plastic clay. The soil in its natural state contains 45% clay sizes, 53% silt sizes, and the remainder sand. After air drying and mixing, random portions were broken down and crushed to pass a No. 40 sieve.

The soil modifications were prepared by washing the soil with 0.75 normal hydrochloric acid to remove the carbonates and sulphates. It was then washed with one normal acetate solution of the desired cation and finally centrifuged with ethyl alcohol. After oven drying at 50^{o}C and crushing, sufficient distilled water was added to yield a slurry with a moisture content slightly in excess of the liquid limit. This slurry was consolidated at a pressure of one kilogram per square centimeter in two-inch inside diameter lucite tubes four inches long. Triaxial specimens, 35.7 mm. by 80 mm. long were trimmed from the consolidated samples. Each specimen contained 5 internal wool wicks inserted just before placing in the triaxial cell.

Soil Tests

The specific gravity of the soil solids, G_s, and the

grain-size distribution (hydrometer analysis) were determined on each modification in accordance with ASTM-D854-52 and D422-54T respectively. The shear strength was determined on a Norwegian Geonor Triaxial Soil Testing Apparatus essentially as outlined by Andresen et al (1). The major difference was the use of a 2 kilogram per square centimeter backpressure for this work.

The quantitative determination of the cation exchange capacity and of the exchangeable cations was according to the procedure evolved by the Soil Survey Section of the Alberta Research Council. The cation exchange capacity was based on leaching a small soil sample with ammonium acetate, then doing a kjeldahl nitrogen determination on the ammonia retained by the soil. The exchangeable cations were determined with a Model DU Beckman Flame photometer on an ammonium leachate of the soil sample.

Test Results

Analysis of Adsorbed Cations

The exchange capacities and the per cent adsorption of the various ions are listed in Table I. One batch of soil provided 3 triaxial specimens. Lack of precise control in leaching and in the final alcohol washing gave rise to the variations in salt content and the adsorbed complex. Group I was the first group tested and Group II the second. The basis for grouping is the difference in the salt contents of the pore water. It is noted that although homionic conditions were not achieved, it was believed that a sufficiently high proportion of a single cation species was adsorbed so that its behaviour would not be affected. Although not fully achieved, the homionic condition was attempted in order to enhance single cation effects and to reduce or eliminate the influence of one cation type on another.

Specific Gravity of Soil Solids

Table 2 presents the results of specific gravity tests as determined by the standard pycnometer procedure. The variation in results is significant particularly when one considers that the natural soil has calcium and magnesium occupying 98% of the exchange positions. The two readily apparent factors influencing these values

are the adsorbed cation complex and the presence of salts in the pore water.

Where soluble salts are present in the pore water, the dry weight of the soil is increased by the weight of salts. For small amounts of soluble salts there is practically no weight loss when they are dissolved in water. If the formula for calculating the specific gravity of the soil solids is considered, the extra weight of salts appears in the numerator but not in the denominator. It is apparent, therefore, that the presence of soluble salts in the pore water increases the observed value of the specific gravity of the soil solids. If the salts are insoluble, they will act like another soil grain and the observed specific gravity will be an average value, either lower or higher depending on whether the salt is lighter or heavier than the soil particles. For most soils this effect will likely be small.

The amount of adsorbed water in the case of sodium soil in the air dry state will be greater than that for calcium in the air dry state because of the greater surface area of the sodium modification. This also leads to an observed specific gravity that is higher in the case of sodium soils than calcium soils.

The influence of the adsorbed cation and the presence of salts in the pore water can not be separated. There is an interplay of these two factors, for example, the presence of salts depress the thickness of the water hull. Since the variations of specific gravity depend to some extent on the surface area and activity of the soil particles larger discrepancies will be found with montmorillonites than with kaolinites.

Hydrometer Analysis

A soil particle in suspension may be considered to consist of a soil nucleus surrounded by a water film. Manipulations of the formulae used in the hydrometer analysis with various assumptions concerning particle size and thickness of adsorbed water film indicate that the influence of the adsorbed cation complex on the water hull of a particle does not have a very significant influence on the resulting grain size curve. The results of tests shown in Fig. 1 reveal, however, a considerable difference of grain distribution particularly for the sodium

modification. Grim (2) suggests that the reason for the increase in the per cent clay sizes is the greater tendency for particles to break down along cleavage planes during dispersion when sodium occupies a high per cent of the exchange positions. The type of clay mineral would reflect to some extent in the grain-size distribution, particularly the montmorillonites.

Triaxial Consolidation

During triaxial consolidation a plot of burette reading versus the logarithm of time was kept for each sample. Table 3 lists the times of theoretical 100% consolidation obtained from these plots. Typical plots are illustrated on Fig. 2.

The results in Table 3 show that there is a cation effect on the time rate of consolidation. The mechanism appears to be a physical clogging of the pore space by the adsorbed water films as is evidenced by the fact that the sodium cation, which is associated with a thick water hull around the particle, yields the modification requiring the longest time. There would appear to be two effects, one decreasing the pore diameter and the other decreasing the number of channels by sealing off those of small diameter. In addition, the breakdown of clay particles when sodium is adsorbed leads to a finer grained soil mass which in turn results in a longer time for consolidation to occur.

Table 3 also shows that there is a noticeable influence exerted on the time rate of consolidation by the presence of salts in the pore water. With the exception of the potassium soil, the higher concentration of salts in the pore water is associated with a decrease in the time required for consolidation to occur. The reason postulated for this is that the salts present in the pore water have the effect of decreasing the thickness of the adsorbed water films on the clay particles and also tend to flocculate the particles. This was substantiated by a single test carried out with sodium modified specimen having a high salt content in the pore water (118 m. e. per 100 gm. air dried soil). The time to theoretical 100% consolidation for this sample was 350 minutes, less than 1% of the time required for the specimen with no salts in the pore water.

Consolidated - Undrained Strength

The data observed for each test consisted of the deviator stress and the pore pressure in the water phase. From this data the major and minor principal and their ratio were calculated. Typical plots of deviator stresses and pore pressure versus strain are shown on Figs. 3 and 4 respectively. From a study of these plates it appears necessary to offer a tentative explanation for the low deviator stresses and pore pressures measured in the sodium modified samples.

The starting point for this tentative explanation is the fact that thick water hulls adsorbed on clay particles are associated with the sodium cation (3). These water hulls must be in contact or somewhat intergrown as is suggested by the time rate of consolidation data. Since the oriented water layers are relatively far from the clay particle surface, the degree of orientation at the fringes is probably not very strong. However, there is some structure in the adsorbed water and in the overlapping portions of adjacent water hulls. The thickness of the water hulls leads to higher void ratios hence higher moisture contents. The water hull thickness also increases interparticle spacing therefore reducing interparticle attractions and repulsions. The monovalent sodium ion is not capable of tying adjacent soil particles together.

The sum of these concepts leads to the conclusion that the shear strength of the sodium modified soils must be low. These effects are almost reversed for the divalent magnesium and calcium cations. A thin water hull around the clay particle is associated with these cations which allows higher interparticle attractions and repulsions and possibly a tying of particles together due to the divalent nature of the cations.

In the consolidated-quick test performed for this report, it was consistently observed that there was a smaller proportion of the applied deviator stress transferred to the water phase of the sodium modified samples than with the other modifications. The suggested explanation for the lower pore pressures developed in the sodium modified soils is developed by the following train of thought. Pressure in the pore water of a soil sample arises because there is a transfer of an applied stress to the water phase from the soil skeleton.

In this context the soil skeleton is meant to include the adsorbed water hull. The transfer of stress to the water phase implies a small decrease in volume of the soil skeleton. From this, it follows that relatively rigid soil structure will result in low pore pressures whereas a soil structure capable of decreasing in volume will result in high pore pressures. It is suggested that the oriented water molecules within the overlapping water hulls impart sufficient rigidity to the soil structure to carry the initially imposed deviator stress. This mechanism also allows for the low deviator stresses observed for the sodium soils since the applied stresses need only disrupt the oriented water net.

The thin and well defined water hulls around clay particles associated with the divalent calcium and magnesium cations do not overlap between soil particles. Therefore, when stress is applied the soil particles can shift or slide over one another sufficiently to bring about a slight volume decrease and to generate the higher pore pressures observed.

The mechanism postulated may also be used to account for the fact that the sodium samples fail at a much lower strain than do the other modifications. Once the initial strain has disrupted the oriented and overlapping water net of the sodium soil, the strength decreases due to the water being in a free state. Particles with large water hulls will slide past one another with relative ease thus internal friction will be at a minimum. On the other hand for the divalent cation modifications, interparticle attractions and repulsions have to be overcome and because the thin water hulls do not smooth out particle irregularities, internal friction can continue to build up to much higher strains.

Mohr Plots

The Mohr Plots of effective stresses series of tests are shown on Fig. 5 and summarized in Table 4. Each envelope is defined by at least three test results except that for the sodium sample with a high salt content in the pore water which is a single test result.

The Mohr plots reveal two distinct facts. One, that the adsorbed cation complex does affect the angle of internal friction and, secondly, that the salt content of the pore or free water has a pronounced effect on the strength of a given modification. At this time

these results must be considered more as qualitative than quantitative due to the uncontrolled salt contents. The general conclusions, however, are still considered to be valid.

The postulates previously presented appear to account for the differences in the angles of internal friction. The potassium and magnesium have relatively thin water hulls which result in lower void ratios and more interparticle interference. The effect is to increase the angle of internal friction. The Mohr envelope for the calcium and sodium modifications offer supporting evidence for the role played by the thickness of the adsorbed water film on the particles. Sodium clay, having the thickest water hull, has the lowest angle of internal friction. The effect of salts in the pore water is also exemplified by the sodium and calcium soils. An increase in salt content results in significant strength increase. The presence of salts has the effect of decreasing the adsorbed water film thickness on the soil particle enhancing interparticle friction. Sodium soils with high salt contents have strengths comparable to the other modifications.

In summary, the variations in strength are attributed to changes in thickness of the adsorbed water films on the clay particles. The thick water films associated with the adsorbed sodium cation tend to overlap between particles which presents a sufficiently rigid soil structure to initial imposed stresses. Due to this structure, a larger proportion of the applied stress is carried by the soil skeleton which is manifested in lower pore water pressures. A small amount of strain disrupts the water nets and accounts for the sodium failing at low strain values. The thick water hulls also smooth out the particles and reduce interparticle friction resulting in low strengths.

Further Considerations of this Work

It has generally been accepted that a plot of the moisture content versus the logarithm of the compressive strength results in a straight line for saturated cohesive soils. The results of these tests have been plotted on Fig. 6. The data strongly suggests that the linear variation only holds when the adsorbed cation complex or the level of salts in the pore water does not change. There is some indication from Fig. 6 that an increase in the salt content flattens the slope of the curve; that is, it tends to impart to the soil those characteristics associated with

a soil of low cohesion. This decrease of cohesion as the salt content increases seems to agree with the behaviour of a clay soil when a non-polar fluid takes the place of water. These results also appear to indicate that the thickness of the adsorbed water film around a clay particle must be taken into account when studying the phenomena of cohesion.

Taylor (4) and others discuss the concept that a plot of the moisture content versus the logarithm of the compressive strength and the virgin compression branch of the pressure-void ratio curve for saturated cohesive soils should result in two parallel lines. The data from this test series and the consolidation data from the work of Hamilton (5) are plotted on Fig. 7. In several instances there is a marked deviation from parallelism of the curves despite the paucity of the data. The major differences between samples is the amount of salts in the pore water and it appears that parallelism will occur only when the soil samples are identical including the salt content of the pore water. At this time there is insufficient data on which to base further conclusions.

The swelling pressure exerted by some soils has presented interesting problems in recent years. In attempting to explain this phenomenon it is suggested that swelling pressures are largely due to increases in thickness of the adsorbed water film. Assuming that reorientation of water molecules in the force field of a clay particle is a crystallization process, then it would appear logical to postulate that this reorientation results in swelling pressures. If one considers a soil mass in a relatively dense state with a high level of salts in the pore water and reasonably pure water becomes available to the mass, it is suggested that the following will take place. Firstly, osmotic pressure will develop due to the dilution of the pore fluid and, secondly, this dilution will allow the adsorbed water films to expand. Both of these mechanisms will give rise to swelling pressures and will continue to do so until an equilibrium is established. If the water available to the soil mass contains salts, both the mechanisms will be reduced in magnitude.

Considering field problems, two major points appear to be the salt level of the groundwater and the composition of water moving through a soil mass. It is possible to decrease the overall salt content of the pore water leading to some strength reduction or it is

possible to affect a cation exchange which may also lead to strength reduction. This latter decrease must be the result of first an exchange of cations, for example, sodium replacing calcium, followed by a reduction of the salt level in the soil mass. In considering the general feasibility of such occurrences in the field, there are two points that must be kept in mind. In many of the soils of Alberta free gypsum crystals are often found. These are sufficiently soluble to maintain a reasonable salt level in the pore water. If the soil is leached with a solution containing divalent and monovalent cations, there is a preferential adsorption of the divalent ion (6). Both these factors mitigate against a strength loss due to cation exchange or salt level decrease.

REFERENCES

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2. R. E. Grim - Applied Clay Mineralogy; McGraw-Hill, 1962.

3. _____ Organization of Water on Clay Mineral Surfaces and Its Implications for the Properties of Clay-Water Systems; Research Board, Special Report No. 40, 1958.

4. D. W. Taylor - Fundamentals of Soil Mechanics; Wiley, 1949.

5. A. B. Hamilton - A Further Study of Cation Effects on the Physical Properties of Clays; M.Sc. Thesis, University of Alberta, 1961.

6. W. P. Kelley - Cation Exchange in Soils; Reinhold, New York, 1948.

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TABLE I

FLAME PHOTOMETER RESULTS

Soil Modification	Exchange Capacity	Group*	Calcium	Percent Sodium	Adsorbed of Potassium	Magnesium	Salts in Pore Water
Natural	28.2	--	84.5	0.5	0.8	14.2	73.6
Potassium	29.0	1	18.6	0.5	75.8	5.1	31.8
	29.6	2	1.7	0.0	95.3	3.0	5.1
Calcium	32.8	1	93.7	0.6	2.1	3.6	21.8
	32.8	2	95.6	0.7	2.5	1.2	9.8
Sodium	32.9	1	5.3	87.9	3.4	3.4	0
	32.9	2	12.6	80.7	3.6	3.2	3.8
Magnesium	32.6	1	9.0	0.6	3.1	87.3	1.8
	32.6	2	5.7	0.0	2.1	92.2	16.5

Notes:

1. *--Group I consists of first 3 samples only for each modification (consolidated at 1.5, 2.75 and 4 kg/cm^2). All subsequent samples comprise Group II.

2. Salts in pore water calculated as difference between exchange capacity by distillation and total of flame photometer results.

3. Exchange capacity and salts in pore water given as milliequivalents per 100 gm air dried soil.

TABLE 2

SPECIFIC GRAVITY OF SOIL SOLIDS

Modification	Average Specific Gravity	Salts in Pore Water m. e. / 100 gm. air dry soil
Natural	2.78	74
Calcium	2.73	10
Magnesium	2.68	16.5
Sodium	2.79	0
Potassium	2.71	32

TABLE 3

TIME IN MINUTES FOR 100% THEORETICAL CONSOLIDATION IN TRIAXIAL CELL

	MODIFICATION							
Cell Pressure	K	S. C. *	Ca	SC	Mg	SC	Na	SC
1.5	9,700	31.8	500	21.8	390	1.8	39,000	0
2.75	14,000	31.8	160	21.8	300	1.8	30,000	0
4.00	13,000	31.8	170	21.8	170	1.8	46,000	0
5.25	3,900	5.1	140	9.8	150	16.5	39,000	3.8
6.5	3,900	5.1	870	9.8	170	16.5	40,000	3.8
7.5	4,500	5.1	175	9.8	150	16.5	22,000	3.8

*SC = Salt content in the pore water, m. e. per 100 gm. of air dried soil

TABLE 4

SUMMARY OF STRENGTH PARAMETERS

Modification	Salt Content of Pore Water m. e. /100 gm. air dried soil	Total Stress C Kg/cm^2	Total Stress ϕ degrees	Effective Stress C Kg/cm^2	Effective Stress ϕ degrees
Natural	74	0.3	11.5	0.2	21
	0	0.3	10.5	0.2	19.5
Potassium	32	0.2	11.5	0.1	23
	5	0.2	13	0.3	23
Calcium	22	0	15	0.1	21
	10	0.4	10	0.5	17.5
Sodium	0	0.1	6	0.2	7.5
	4	0	10	0	14
	118	0*	15*	0*	30*
Magnesium	2	0	14	0.1	24
	16	9	14	0	24

* Single test result, cohesion assumed equal to zero

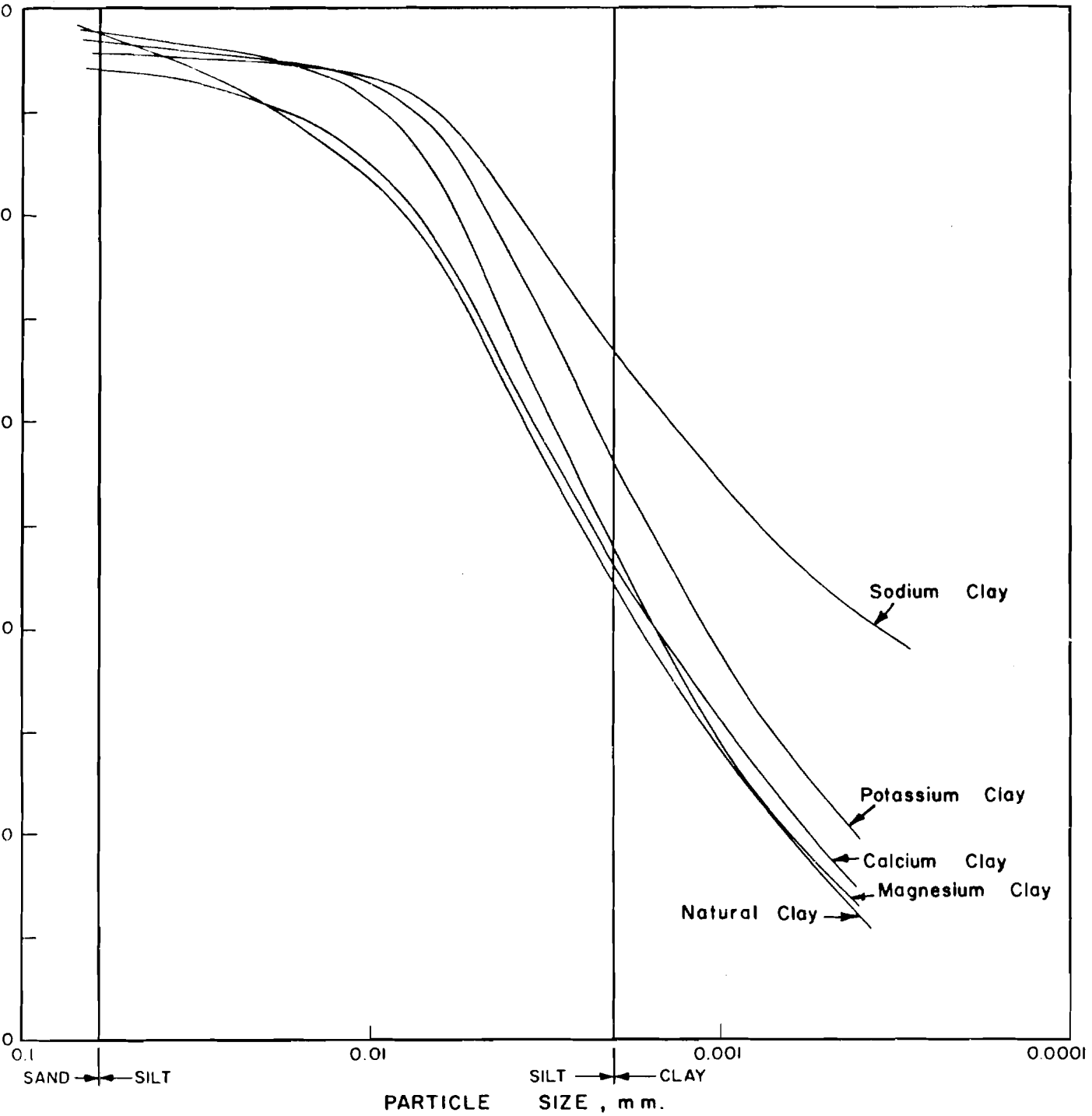


FIGURE I

PARTICLE SIZE DISTRIBUTION OF THE CLAY MODIFICATIONS

by hydrometer analysis

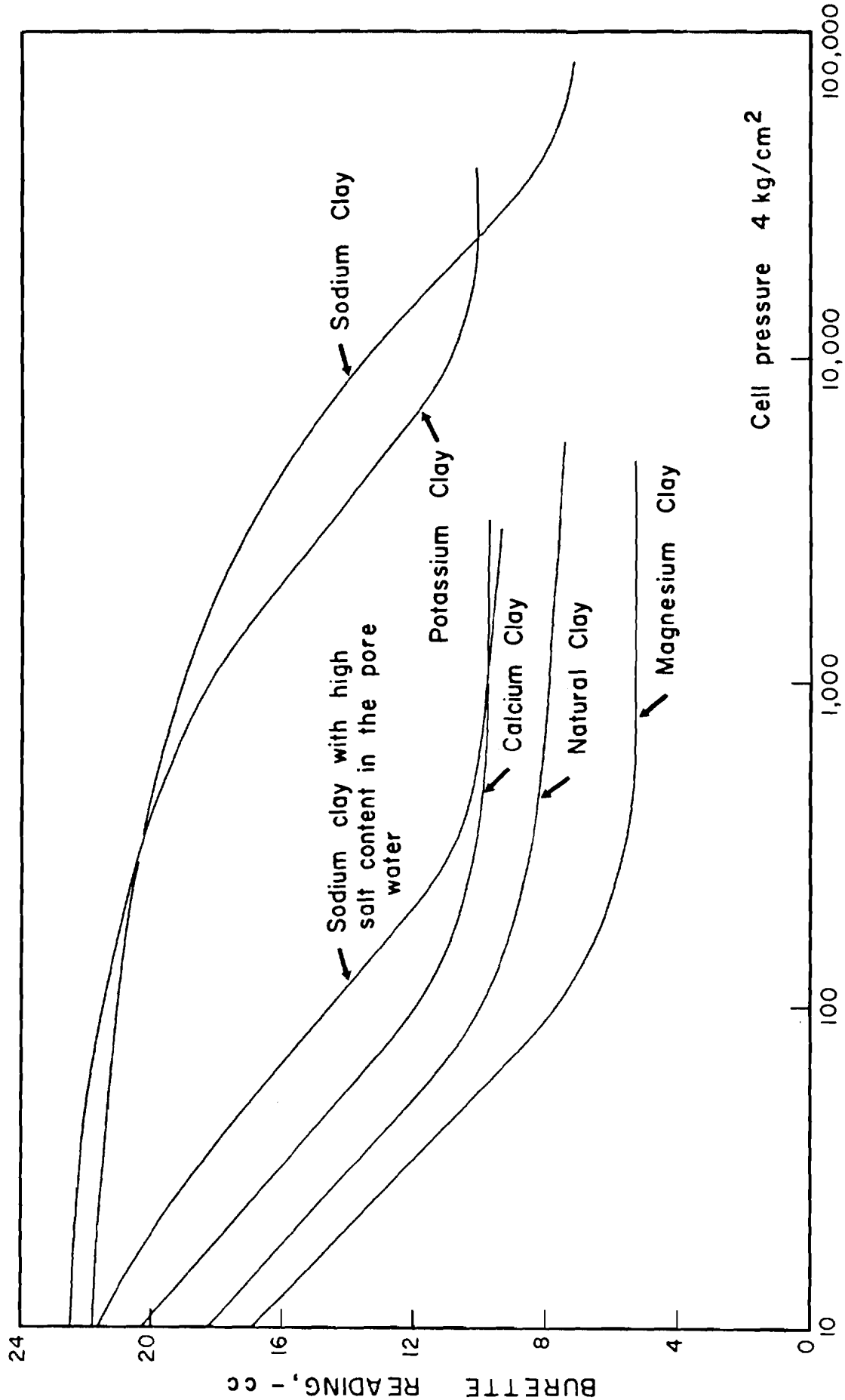


FIGURE 2 TRIAXIAL CONSOLIDATION CURVES FOR THE CLAY MODIFICATIONS

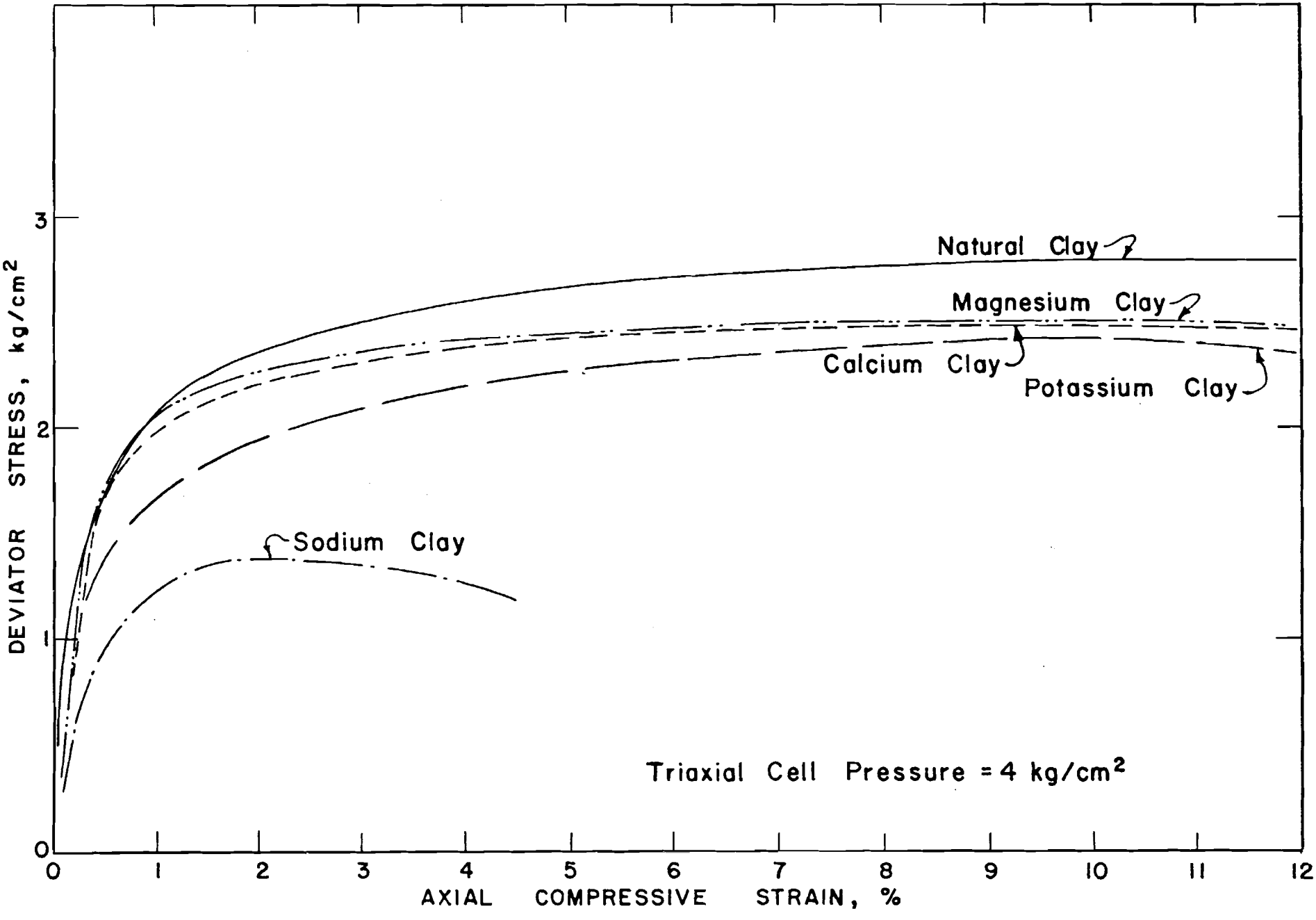


FIGURE 3 STRESS-STRAIN RELATIONSHIPS FOR THE CLAY MODIFICATIONS

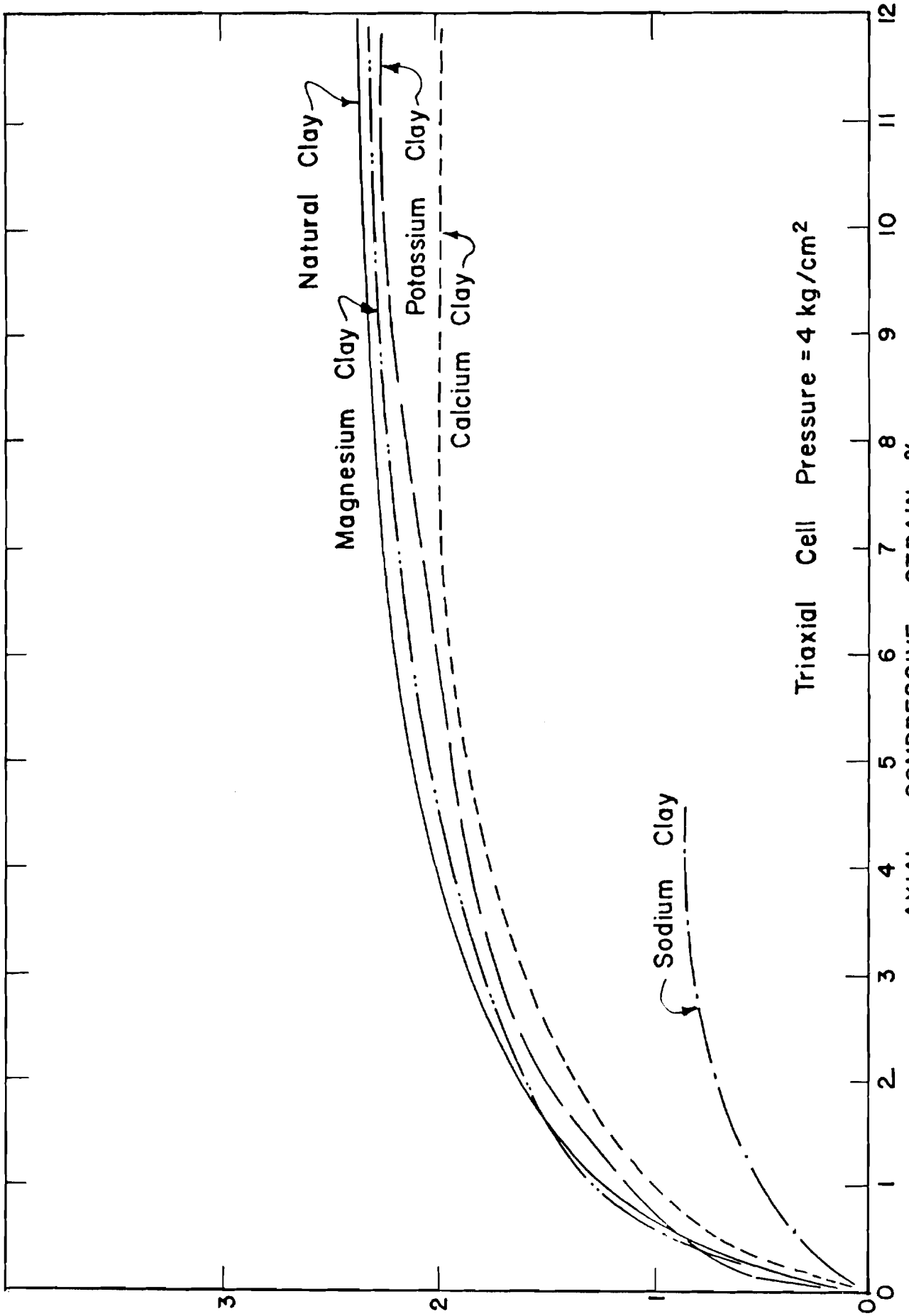
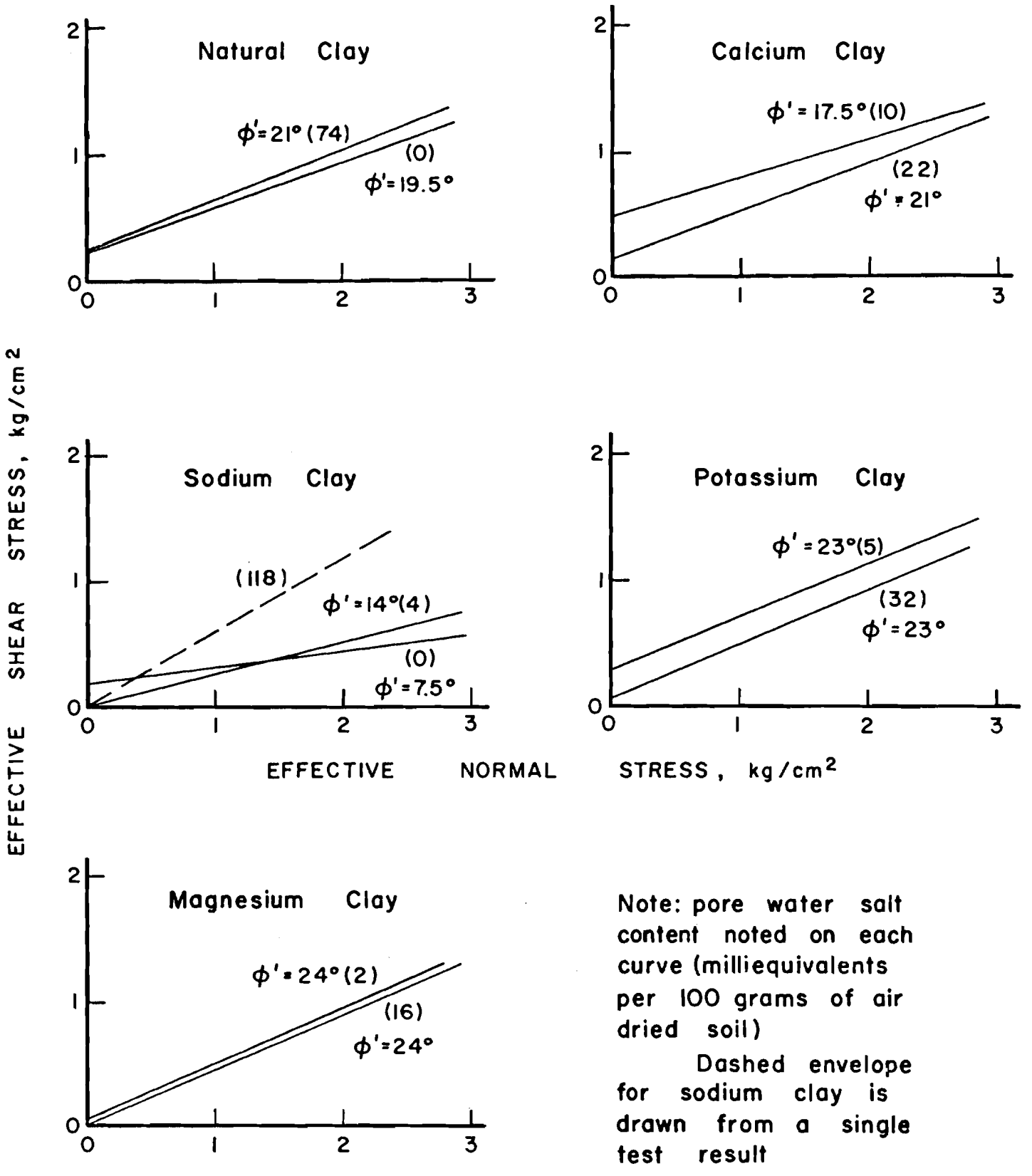


FIGURE 4 PORE PRESSURE–STRAIN RELATIONSHIPS FOR THE CLAY MODIFICATIONS



Note: pore water salt content noted on each curve (milliequivalents per 100 grams of air dried soil)

Dashed envelope for sodium clay is drawn from a single test result

FIGURE 5 MOHR ENVELOPES FOR THE CLAY MODIFICATIONS

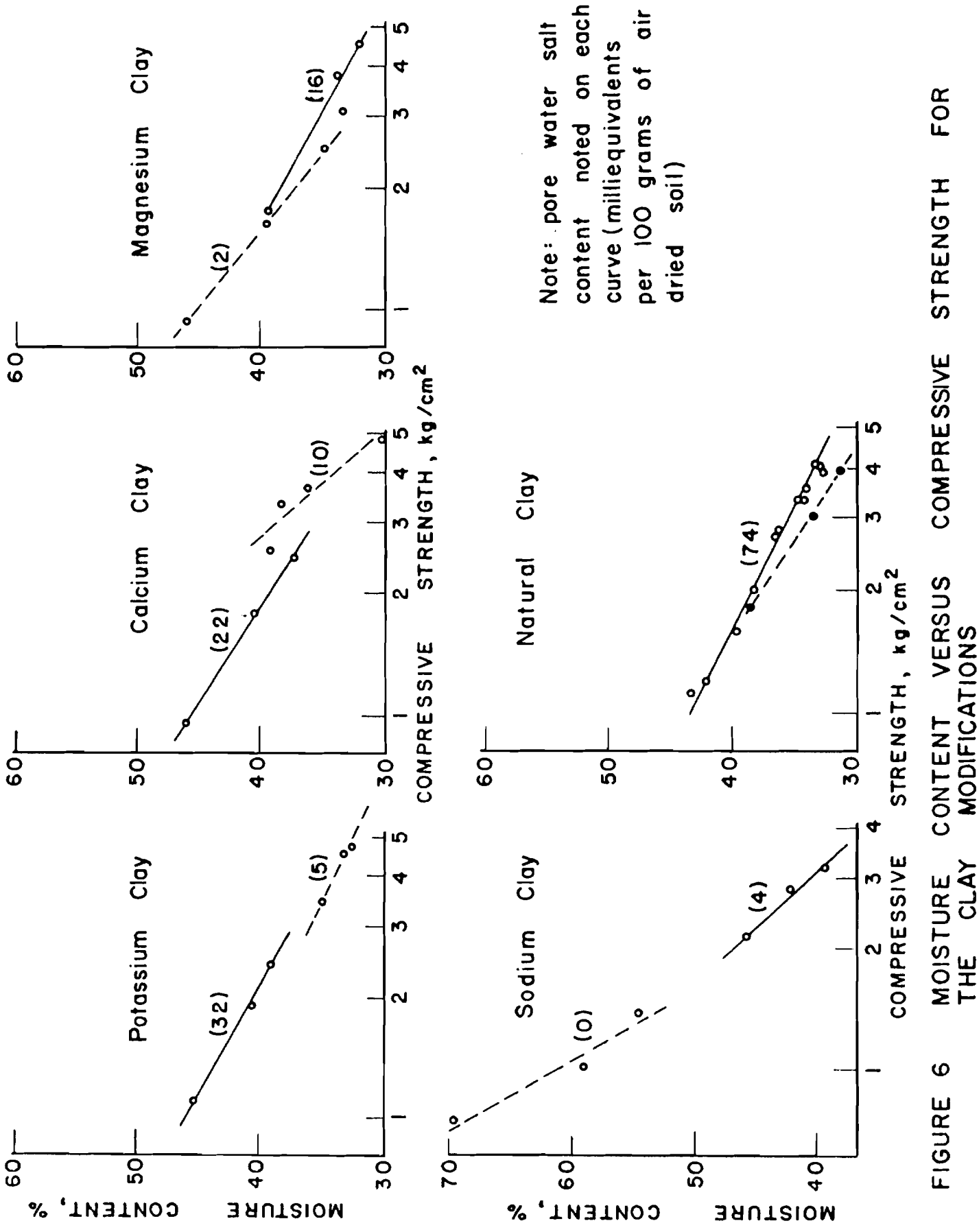
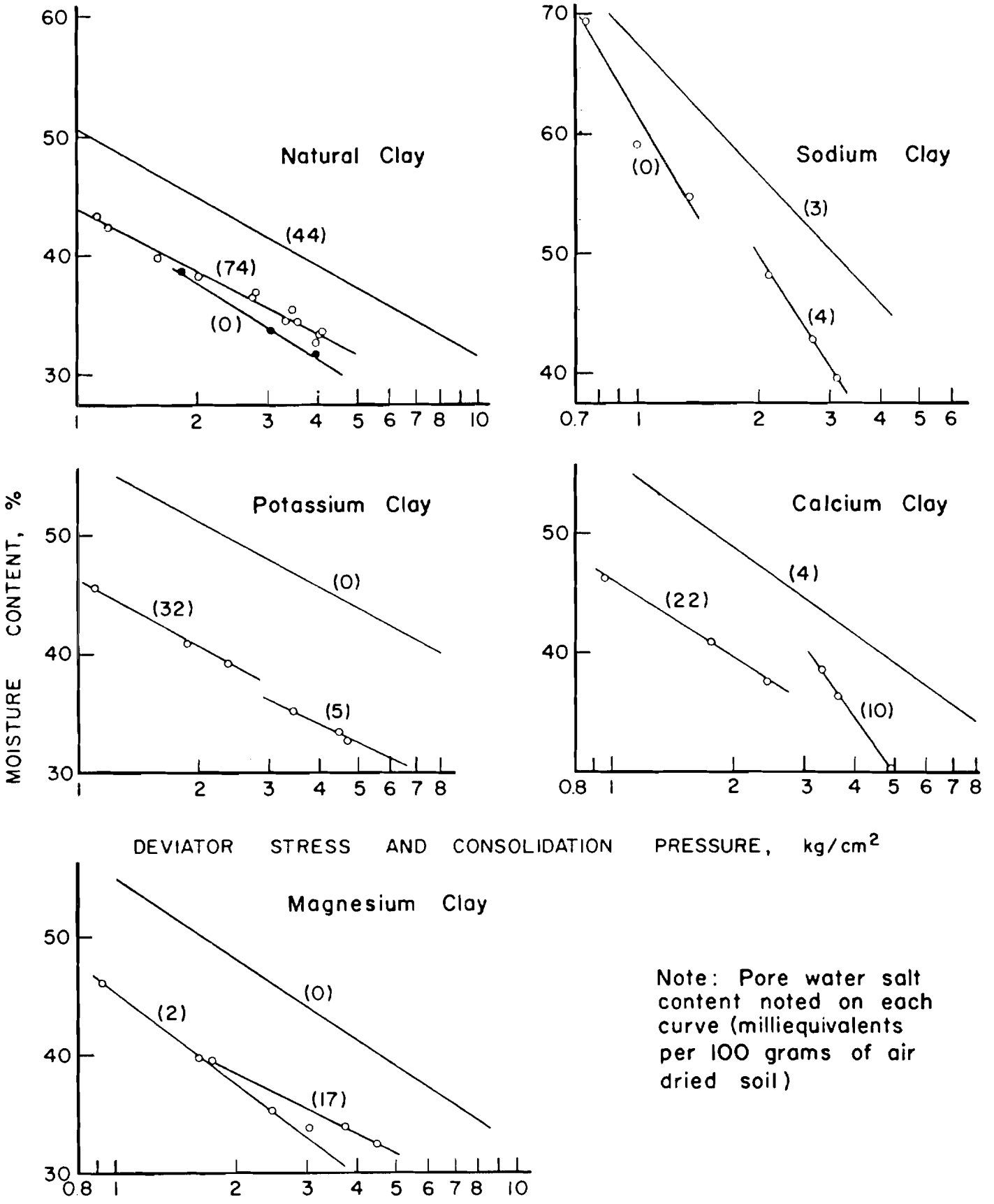


FIGURE 6 MOISTURE CONTENT VERSUS COMPRESSIVE STRENGTH FOR THE CLAY MODIFICATIONS



Note: Pore water salt content noted on each curve (milliequivalents per 100 grams of air dried soil)

FIGURE 7 RELATIONSHIP BETWEEN MOISTURE CONTENT, VIRGIN COMPRESSION BRANCH AND DEVIATOR STRESS FOR THE CLAY MODIFICATIONS

THE ENGINEERING PROPERTIES OF ILLITE RELATED TO FABRIC AND PORE WATER COMPOSITION

by

R. M. Quigley

SUMMARY

Engineering tests have been performed on an illite clay containing about 30 per cent of vermiculite and chloritized vermiculite. The tests were performed in controlled salt environments consisting of NaCl and KCl. The clays were sodium saturated, deposited in 0.154 Normal NaCl and then leached with 0.154 Normal solutions of either Nacl or KCl. Different consolidation and strength properties were observed on the clay depending on whether it was leached with NaCl or KCl. A fabric study, performed using x-ray diffraction techniques, showed that the Na and K clays had different average particle arrangements. Most of the differences in soil fabric and engineering properties of the Na and K clays can be attributed to the different adsorbed cations present. Although a limited amount of potassium fixation and layer lattice collapse did occur in the soil vermiculite when leached with KCl, the effects of the resulting expulsion of interlayer water were generally swamped by the effects of ordinary cation exchange.

- * -

During acid weathering, considerable interlayer potassium may be removed from soil illites or micas producing "degraded" illites or vermiculites which can expand appreciably and may behave somewhat like montmorillonite. Upon treatment with KCl solutions they collapse back to a non-expanding mica. This collapse occurs in nature when degraded illites fix potassium upon entering a marine environment. The reaction has been called clay mineral "diagenesis" (change in mineralogy after deposition) by some writers and cation exchange with fixation by others. (eg. Weaver, 1958a and 1958b; Powers, 1957).

The purpose of the research described here was to determine the relative effects of potassium fixation and ordinary cation exchange on the engineering properties of a potassium deficient illite in controlled solutions of NaCl and KCl. The degraded soil illite used in most of the experiments was obtained from a Podzolic soil profile in which acid weathering had reduced the potassium content to a fairly low level. Although a minor amount of collapse did occur as a result of KCl leaching, the soil behaved essentially like an illite and the changes in properties were due primarily to ordinary cation exchange. The collapse of the clay was prevented by adsorbed aluminum and iron hydroxide complexes which had replaced the potassium resulting in partial chloritization of the degraded illite or vermiculite in the soil profile. Different consolidation and strength characteristics which were measured on the Na and K clays, are the subject of this paper.

Soil fabric studies performed on consolidated specimens showed that the Na and K clays had different particle arrangements which could be measured semi-quantitatively by x-ray methods. Additional studies on sheared illite and kaolinite specimens showed that shearing produces marked particle orientation. The differences in soil fabric and engineering properties of the illite can be explained in terms of Rosenqvist's concepts of cation polarizability or related to the size of the hydrated cations adsorbed on the clay.

MATERIALS AND METHODS

Clay Mineralogy

The clay soil used for testing was obtained from a

Podzolic soil profile located near the crest of a drumlin in East Boston, Massachusetts. The clay was separated from the till by high speed centrifugation. The soil came from the upper 1.5 feet of the profile where the measured pH of the soil water was 4 to 4.5. The K_2O content of the clay was 3 to 4 per cent or about half that of normal illites.

The soil mineralogy was studied in detail using a variety of methods including x-ray diffraction, differential thermal analysis and glycol retention. X-ray spectographic procedures were used to determine the cation exchange capacity, total potassium, and extractable aluminum and iron. Details of the clay mineralogy and the mineralogical test procedures have been published elsewhere (Quigley and Martin, 1961) and will not be repeated here. Although several batches of clay were tested, all showed essentially the same properties despite large differences in grain size. Batches 2 and 3 mentioned here, consisted of sizes smaller than 2 microns and 0.4 microns, respectively. A brief description of the composition of the clay used for engineering testing follows.

The clay was composed chiefly of illite with up to 30 per cent of chlorite and soil vermiculite. A typical x-ray diffraction trace of an oriented specimen homionic in Na is shown on Fig. 1. The strong 10.1 angstrom peak at the left side of Fig. 1 is characteristic of illite. The 14.3 and 7.4 angstrom peaks indicate the presence of chlorite and vermiculite. Heat treatment and sodium citrate extraction studies indicated that iron and aluminum chlorites were both present. Although potassium saturation caused the vermiculitic phase to collapse back to 10 Å illite, the change was not permanent and the potassium could generally be easily exchanged by sodium. The cation exchange capacity of the clay varied from 20 to 35 milli-equivalents per 100 grams of clay, depending on the size of the clay fraction and its location in the soil profile. Aluminum and iron hydroxide complexes were found to have chloritized much of the soil vermiculite thus preventing collapse of the structure (Quigley and Martin, 1961). Of particular practical interest was the fact that extensive leaching with solutions of NaCl, and KCl in particular, resulted in the removal of some of the adsorbed Al and Fe hydroxide complexes and an increase in cation exchange capacity. Any long-term increase in exchange capacity would probably have affected the engineering test results, so all tests, with the exception of consoli-

dation 34 were carried out over the same period of time.

Methods of Engineering Testing

The clay samples were sodium saturated by washing them several times in 1 Normal NaCl solutions. They were then brought into equilibrium with 0.154N NaCl and deposited in 0.154N NaCl solutions. One part per million mercuric chloride was used to prevent any biological activity from developing in the clay sediment. The samples were allowed to consolidate in the consolidation chamber under their own weight and then under the weight of a light porous stone. All samples were then leached under the same head with 0.154N solutions of either NaCl or KCl. During the 8 or 9 day leaching period, 2 to 3 soil volumes of leachate passed through the sedimented samples. The leached clays were subsequently consolidated to pressures of up to 8 tons per square foot. Permeability values were determined from both the consolidation test results and from falling head tests run on the clay at the end of the load increments. Temperature effects were eliminated by running the consolidation tests in a constant temperature room.

The consolidated clay samples were tested in undrained shear using a specially built miniature vane turned with the aid of a sensitive Waters torque glass. Strain was not measurable with the apparatus employed and failure was reached in about one minute.

The pore fluid in the consolidated clay samples was analyzed for sodium and potassium using a flame photometer.

The clay remaining after the vane testing was completed, was remoulded and tested in drained shear using a direct shear machine. The K:Na ratio was found to have such an important effect on the drained strength that several samples were washed in KCl to remove all of the adsorbed sodium prior to testing.

Fabric studies were run on carefully prepared air dried clay specimens using x-ray diffraction techniques described previously (Quigley 1961). Briefly, the method consisted of carefully grinding a flat surface on a block of air dry clay cut from an engineering test specimen. The block was then mounted in the diffraction machine and the prepared surface was x-rayed. Successive layers could be

ground off the specimen so that a series of surfaces passing through a shear failure zone could be x-rayed. The degree of orientation of the clay platelets could be semi-quantitatively determined by comparing the height of the (00ℓ) and $(hk\ell)$ x-ray peaks obtained on surfaces of equal area.

TEST RESULTS

The results of the engineering tests and the fabric study are presented under separate headings below. The chemical data required for control are presented with the engineering results.

ENGINEERING STUDY

Index Tests

Atterberg limits run on the clay showed that KCl leaching caused both the liquid and plastic limits to increase. Typical results were obtained from Batch 3 on which liquid limits of 99 and 120 were obtained for the Na and K clays, respectively. The corresponding plastic limits were 40 and 47, respectively.

Consolidation Tests

Typical consolidation curves are shown on Figs. 2 and 3 for the Batch 2 and Batch 3 clays, respectively. It is obvious that the KCl leached clays exist at a higher void ratio than the Na clays at any consolidation pressure within the range studied. The greatest spread in the curves occurs for those clays which were most highly leached and hence had the highest K: Na ratios. The K:Na ratios were determined after the consolidation tests were completed by measuring the K and Na present in the pore fluid of samples cut from the test specimens. The ratios are recorded on the figures.

Strength Tests

Undrained and drained shear strength envelopes for the clay from Batch 3 are shown on Fig. 4. Any differences in the undrained vane strengths of the Na and K clays, if present, are too small to be of significance. The drained tests, on the other hand,

showed pronounced changes depending on the K:Na ratio. A drained friction angle of 16 degrees was measured on the Na clay. This angle increased to 20 degrees for a partially leached clay having a K:Na ratio of 1.5:1. When the clay was washed with KCl until all of the Na had been removed, the friction angle increased to 23 degrees. The corrected position of the middle point on this steeper envelope conforms with several other envelopes and semi-log plots of the data. A similar increase in drained friction angle was observed for the coarser clay from Batch 2. In this case, ϕ_d increased from 20 degrees for Na clay to 26 degrees for the same clay with a K:Na ratio of 2.9:1. In Fig. 5, the drained shear strengths are plotted against water content. This plot shows even more strikingly the magnitude of the changes and the significance of the K:Na ratio. The K clays are considerably stronger than the Na clays, the strength increasing as the K:Na ratio increases. It should also be mentioned here that the undrained vane strengths of the K clays were greater than the Na clays at all water contents within the range studied. The undrained strength differences were not nearly as pronounced or consistent as those of the drained tests and are believed to have been affected by clay particle orientation in the consolidated specimens tested (see Fabric Study).

FABRIC STUDY

Maximum and Minimum Parallelism

Maximum parallelism of the clay platelets was obtained by high speed centrifugation of a dispersed clay suspension onto a porous ceramic plate. An x-ray diffraction trace of a highly oriented, glossy layer of kaolinite is shown by curve (a) on Fig. 6. Minimum parallelism or a net randomness was obtained by x-raying a level surface of clay powder. A typical diffraction trace is shown by curve (d) of Fig. 6. The (00ℓ) and ($hk\ell$) peaks shown on curve (d) are quite small and about the same size. Particle parallelism greatly enhances the size of the (00ℓ) peaks as shown by curve (a). Similar curves were obtained for the illites tested.

Anisotropically Consolidated Illite

Small rectangular blocks, all but consolidation 31 having top and side areas within 3 per cent of each other, were cut

from the consolidated samples of consolidation tests 31 through 35 shown on Fig. 3. One top and side surface of each block was ground flat and x-rayed. An approximate arithmetical correction for the smaller area was applied to test 31. In Fig. 7 the height of the (003) basal peaks from the top surfaces and the height of the 4.48 Å(hk0) peaks from the side surfaces are plotted against the maximum pressure to which the samples were consolidated. Although the data is rather meagre, it does appear as though the Na clays have the larger x-ray peaks at a given pressure. This indicates that they have been more highly oriented perpendicular to the direction of the consolidation pressure than the K clays. In other words, the K clays have a more open structure hence their higher void ratio at a given pressure. The anomalous position of consolidation 34 is probably related to the fact that it was the only specimen recycled and subjected to two intervals of leaching. As mentioned previously, extended leaching periods caused the cation exchange capacity to increase as strongly adsorbed Al and Fe hydroxide cations were slowly removed.

Fig. 8 illustrates the particle orientation within a sheared drained direct shear sample which was air dried subsequent to testing. The test conditions are shown on the Figure. It can be seen that a marked parallelism has developed due to shear near the bottom of the sample. The corresponding water content in this zone of shear was 65.7 per cent compared to 66.8 per cent near the top. Visual examination of the dried direct shear specimens indicated that the failure plane was always arcuate in shape, curving either towards the top or the bottom in order to conform with the direction of the principle stresses at the edges of the shear box. On the basis of the x-ray results the plots of water content versus drained strength were prepared using the lowest of the three moisture contents measured rather than an average for the whole sample. The selected values produced much smoother curves than did the average water contents.

Although not related to the illite study, an analysis of the fabric in the failure zone of a triaxial sample of kaolinite was of considerable significance and is worth describing here. An air dried triaxial sample of highly flocculated kaolinite (smaller than 2 microns) was obtained from A. Wissa (1961). The specimen had been failed in undrained shear over a period of 50 hours. The shear displacement

was of the order of 2 millimeters and occurred in a shear zone about 2 millimeters wide. Sharply defined slip surfaces were clearly visible.

According to Wissa, (personal communication) about 80 per cent of the excess pore pressure built up during shear was retained in the sample after the deviator stress was completely removed. This can be explained by hypothesizing particle reorientation inside the failure zone of the flocculated sample. Bjerrum, 1961, described a series of tests on extremely sensitive Norwegian quick clays which showed similar results. He hypothesized particle reorientation in the failure zone to explain his high "A" factors and his high residual pore pressures after removal of the deviator stress.

The results of x-ray diffraction tests on Wissa's sample are plotted in Fig. 9. The very pronounced increase in height of the 7Å (001) kaolinite peak indicates marked particle orientation within the shear zone. The plot of the 4.35A (hk0) peak also shows a slight increase in height. This may be due to an increase in density of the kaolinite within the shear zone after air drying. The greater parallelism permits greater shrinkage and hence a greater dry density.

In Fig. 6, four diffraction traces of the same Georgia kaolin are illustrated. Curve (a) represents maximum parallelism produced by centrifugation as described previously. Curve (d) is a powder pattern representing complete randomness. Curves (b) and (c) were obtained inside and outside of the shear zone, respectively. In all cases, the area of sample exposed to x-rays was the same. It is obvious that although considerable particle orientation has been produced by the shearing, it is not great when compared with curve (a).

DISCUSSION

The increase in the liquid limit of illite, produced by increasing the size of the adsorbed monovalent cation has been described by Rosenqvist, 1957. The results of his tests on both illite and montmorillonite are shown on Fig. 10. He relates the increase in liquid limit to the increasing polarizability of the cations as their size increases. The large cations are more easily distorted into an induced dipole by

by the force fields between the clay particles, than the smaller cations.

Rosenqvist, 1955, and Moum and Rosenqvist, 1961, suggest that the polarizable cations induce an increased attraction between particles. More water would, therefore, be required in a potassium clay to reduce the interparticle forces to the same level as in a sodium clay. If we assume that the net forces at the liquid limit are the same in Na and K clays (i. e. they have the same strength), the K clay will have the higher moisture content (higher liquid limit).

Another equally plausible explanation is that the sodium cation, which has a higher ionic potential and a larger hydrated radius than potassium, may have a lubricating effect on the soil system. The larger size of the hydrated sodium ions may prevent the clay particles from approaching as close to one another thus lowering the forces of attraction and reducing the shear strength of the clay at a given water content.

Moum and Rosenqvist, 1961, explain the shape of the montmorillonite curve in Fig. 10 as being a function of the amount of interlayer (infracrystalline) water. For K, Rb and C_s the amount of interlayer water is about the same so that the liquid limit is a straight line function of the log polarizability as in illite. On the other hand Li and Na absorb a great deal of water because they have very high ionic potentials. This gives these cations very large hydrated radius and the montmorillonite swells to large "d" values. In other words, the montmorillonite swells until it contains a great deal of interlayer water which is represented by the hatched area in Fig. 10.

A "beidellite" montmorillonite from California, saturated with Na and K, gave results similar to Rosenqvist's montmorillonite. The illite clay studied here, despite a small content of collapsible vermiculite, behaved like Rosenqvist's illites.

Moum and Rosenqvist, 1961, also describe a group of experiments designed to study the effects of interlayer water and interparticle water on the shear strength of sodium and potassium clays. Illites and montmorillonites, artificially sedimented and consolidated in 35 parts per thousand NaCl, were subjected to KCl leaching for a period of 26 months before strength testing. The illites which were leached

with KCl had undrained strengths about 1.6 times greater than the corresponding Na illites at given consolidation pressures. The differences in properties once again were explained in terms of the polarizability of the adsorbed cations. With the more polarizable potassium ion present, the attractive forces and bonds between the clay particles of the flocculated system are increased.

The results of the consolidation fabric studies can be explained by either Rosenqvist's concepts of cation polarizability or by concepts of ionic potential and the size of the adsorbed hydrated cations. The K clays were certainly more resistant to compression as indicated by both the consolidation curves and the x-ray fabric study. The drained strength results also are readily explained by either concept. Very little difference was observed in the undrained strengths of the Na and K illite studied here. The reason is probably the very different stress history of samples. The clays tested by Moum and Rosenqvist were leached after consolidation whereas those tested here were leached and then consolidated.

CONCLUSIONS

1. The clay soil tested consisted of illite with up to 30 per cent of vermiculite and chloritized vermiculite.

2. Different engineering properties were obtained on the clay, depending on whether Na or K was the adsorbed cation. In general, the K clays were stronger and more resistant to consolidation than the Na clays.

3. An x-ray fabric study on consolidated clay samples showed that the KCl leached clays had a more flocculated or open structure than the NaCl leached clays. In other words, the Na clays had developed more parallelism during consolidation than the K clays.

4. The engineering test results and the fabric data may be explained in terms of Rosenqvist's concepts of cation polarizability or in terms of ionic potential and the size of the hydrated radii of the adsorbed cations.

5. Continuous salt water leaching removed some of the complex aluminum and iron hydroxide cations adsorbed on the clay, resulting

in increases in the cation exchange capacity and changes in the soil properties. Considerable chemical control is required in all long-term tests on clay soils in order to interpret the engineering test results.

6. The results of the preliminary fabric study described here suggest that x-ray techniques could become a very useful tool in measuring the average particle arrangements of the clay platelets in a soil mass.

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10. A. E. Z. Wissa - A Study of the Effects of Environmental Changes on the Stress-Strain Properties of Kaolinite, S. M. Thesis, Mass. Inst. of Technology, 1961.

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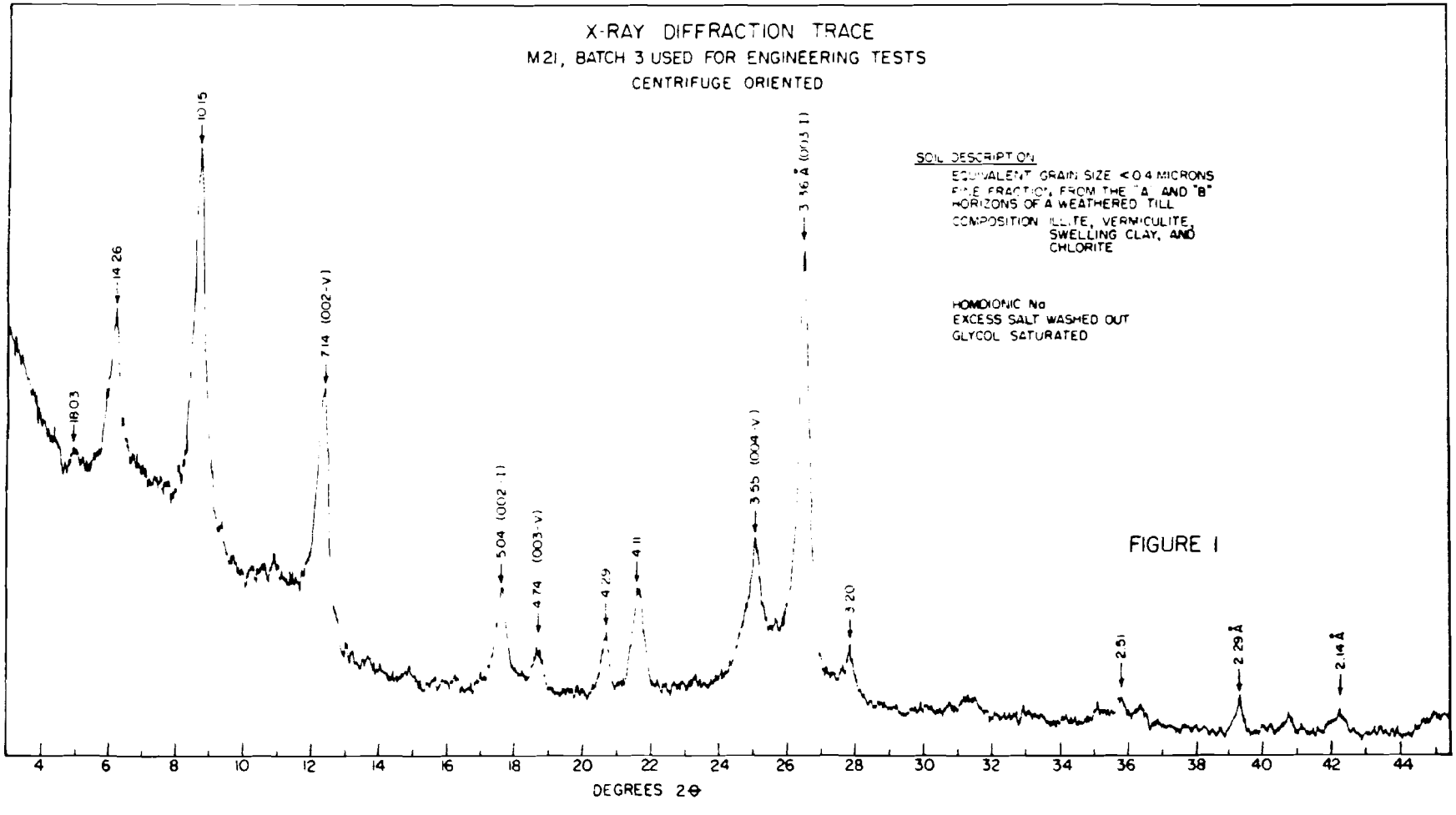
X-RAY DIFFRACTION TRACE
M21, BATCH 3 USED FOR ENGINEERING TESTS
CENTRIFUGE ORIENTED
SOIL DESCRIPTION
EQUIVALENT GRAIN SIZE < 0.4 MICRONS
FINE FRACTION FROM THE "A" AND "B"
HORIZONS OF A WEATHERED TILL
COMPOSITION ILLITE, VERMICULITE,
SWELLING CLAY, AND
CHLORITE
HOMOIONIC Na
EXCESS SALT WASHED OUT
GLYCOL SATURATED
18.03
14.26
10.15
7.14 (002-V)
5.04 (002-I)
4.74 (003-V)
4.29
4.11
3.55 (004-V)
3.36 Å (003-I)
3.20
2.51
2.29 Å
2.14 Å
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
DEGREES 2Θ

FIGURE 1

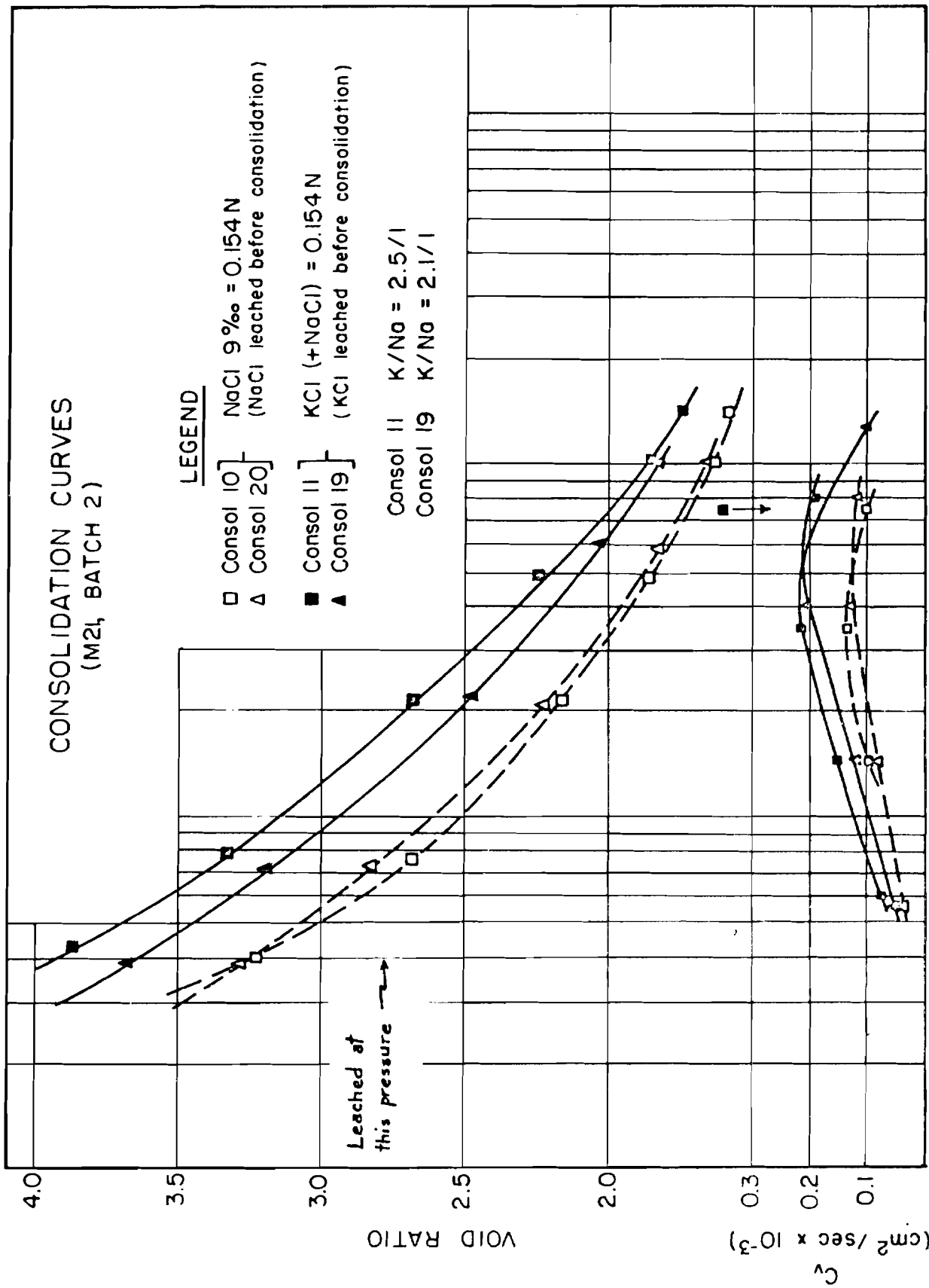
CONSOLIDATION CURVES
(M24, BATCH 2)
LEGEND
Consol 10 NaCl 9‰ = 0.154 N
Consol 20 (NaCl leached before consolidation)
Consol 11 KCl (+NaCl) = 0.154 N
Consol 19 (KCl leached before consolidation)
Consol 11 K/Na = 2.5/1
Consol 19 K/Na = 2.1/1
Leached at this pressure
VOID RATIO
4.0
3.5
3.0
2.5
2.0
C_v (cm^2/sec x 10^{-3})
0.3
0.2
0.1
10
100
1000
10,000
CONSOLIDATION PRESSURE (psf)

FIGURE 2

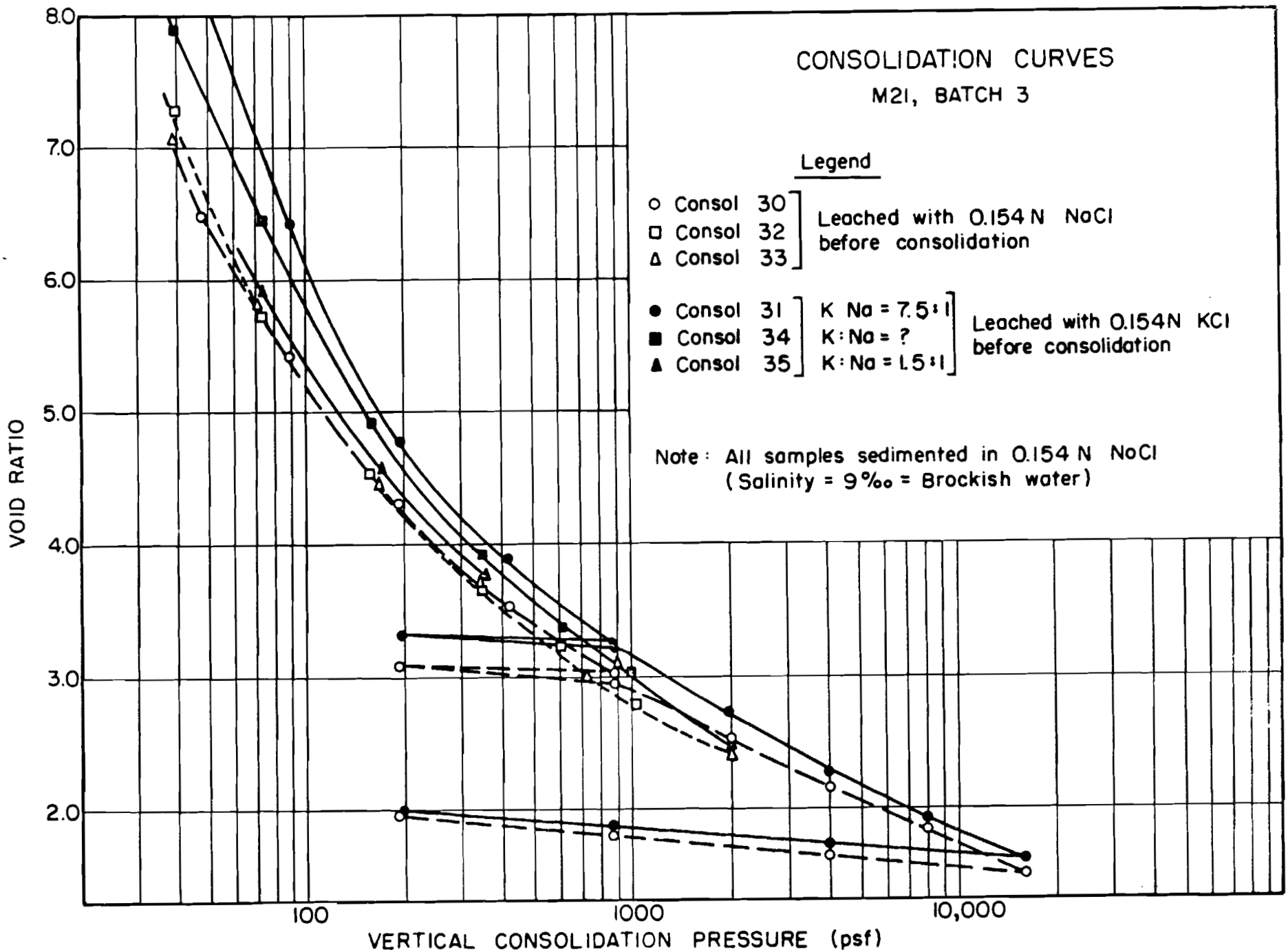
CONSOLIDATION CURVES
M21, BATCH 3
Legend
Consol 30
Consol 32
Consol 33
Leached with 0.154 N NaCl before consolidation
Consol 31
Consol 34
Consol 35
K Na = 7.5 : 1
K : Na = ?
K : Na = 1.5 : 1
Leached with 0.154N KCl before consolidation
Note : All samples sedimented in 0.154 N NaCl (Salinity = 9‰ = Brockish water)
VOID RATIO
8.0
7.0
6.0
5.0
4.0
3.0
2.0
100
1000
10,000
VERTICAL CONSOLIDATION PRESSURE (psf)

FIGURE 3

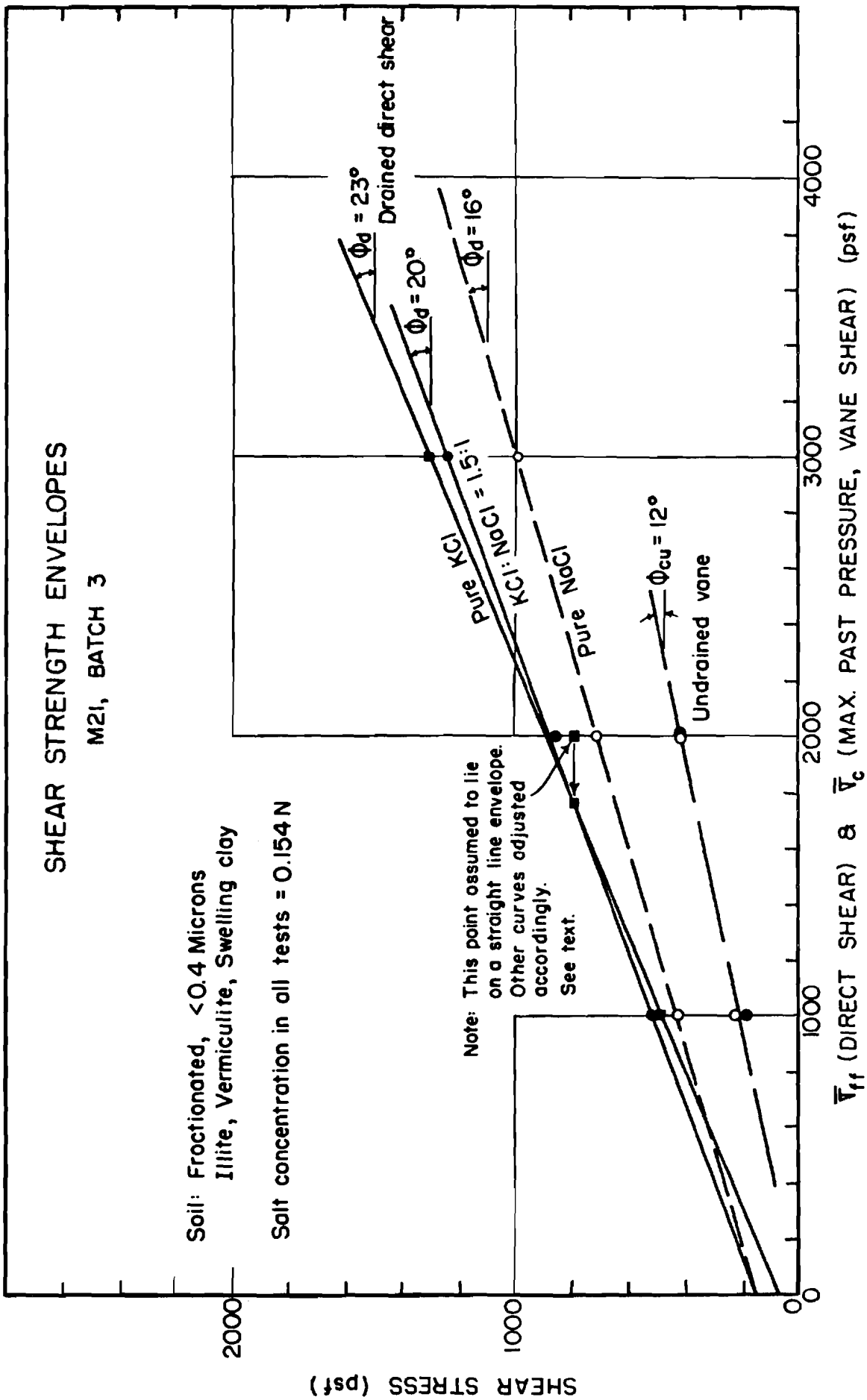
SHEAR STRENGTH ENVELOPES
M21, BATCH 3
Soil: Fractionated, <0.4 Microns
Illite, Vermiculite, Swelling clay
Salt concentration in all tests = 0.154 N
Note: This point assumed to lie on a straight line envelope. Other curves adjusted accordingly. See text.
Pure KCl
KCl: NaCl = 1.5:1
Pure NaCl
Undrained vane
Drained direct shear
SHEAR STRESS (psf)
0
1000
2000
3000
4000
$\bar{\tau}_{ff}$ (DIRECT SHEAR) & $\bar{\tau}_c$ (MAX. PAST PRESSURE, VANE SHEAR) (psf)

FIGURE 4

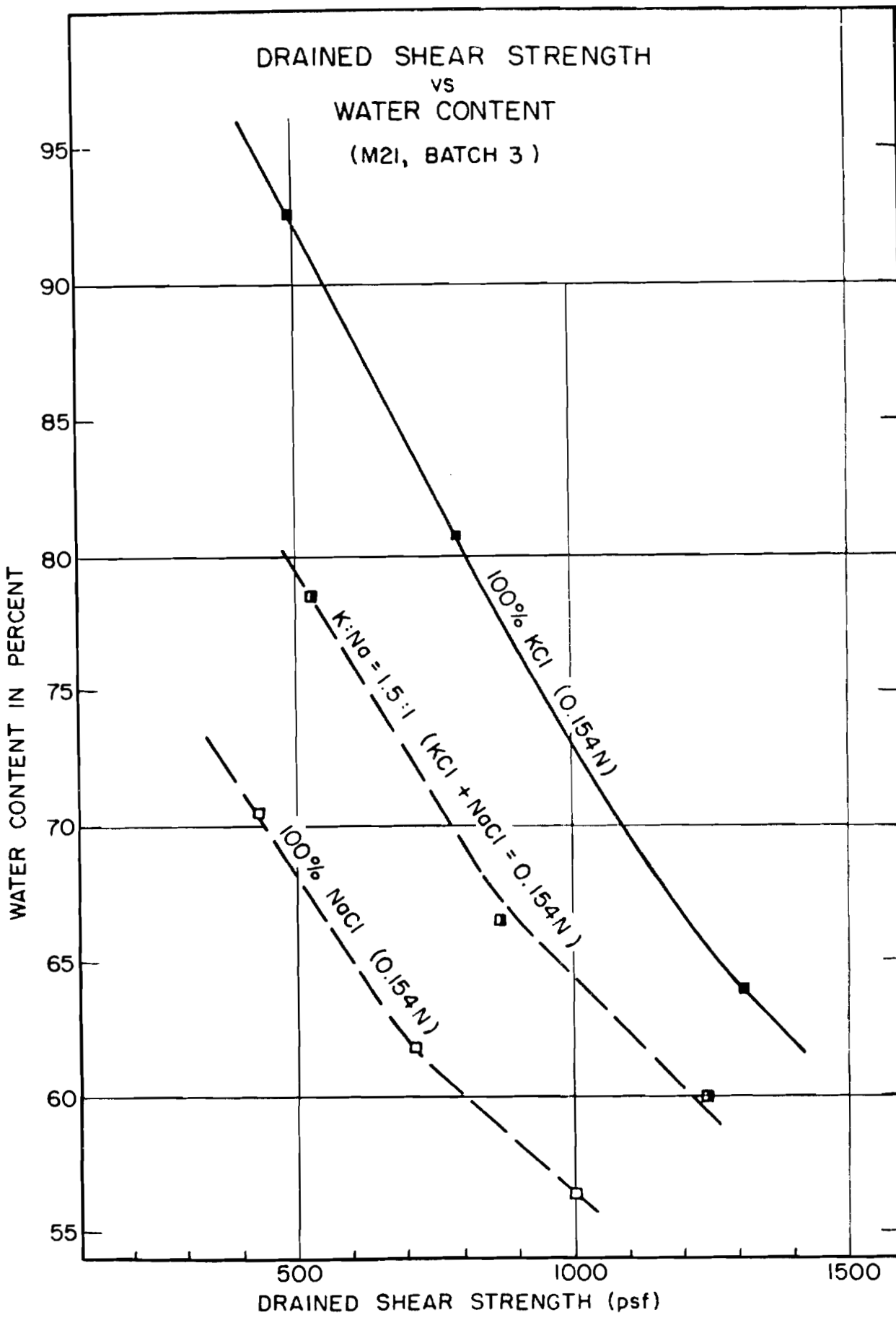
DRAINED SHEAR STRENGTH
VS
WATER CONTENT
(M21, BATCH 3)
WATER CONTENT IN PERCENT
95
90
85
80
75
70
65
60
55
500
1000
1500
DRAINED SHEAR STRENGTH (psf)
100% KCl (0.154N)
K:Na = 1.5:1 (KCl + NaCl = 0.154N)
100% NaCl (0.154N)

FIGURE 5

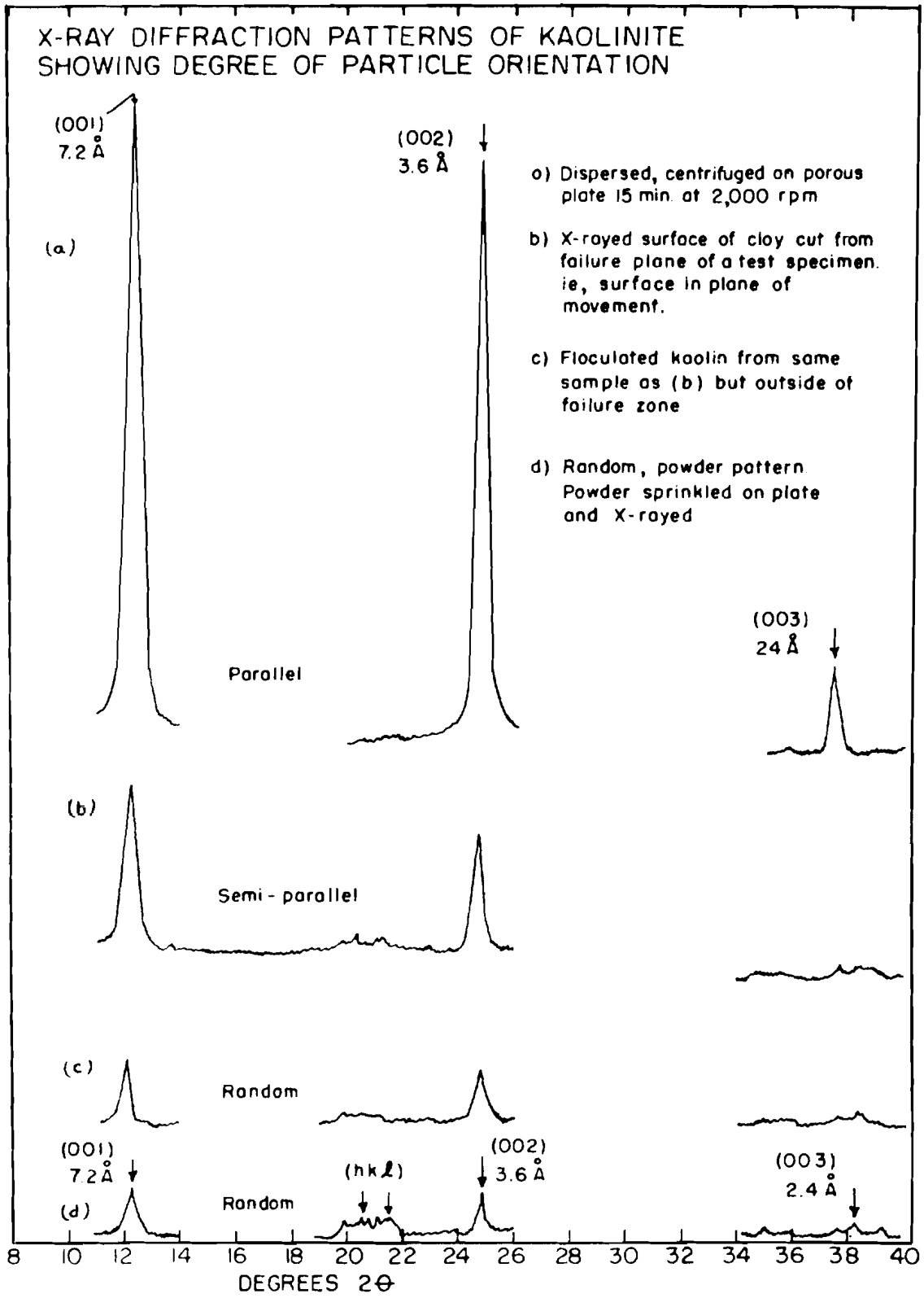
X-RAY DIFFRACTION PATTERNS OF KAOLINITE
SHOWING DEGREE OF PARTICLE ORIENTATION
(001)
7.2 Å
(002)
3.6 Å
(a)
a) Dispersed, centrifuged on porous plate 15 min. at 2,000 rpm
b) X-rayed surface of cloy cut from failure plane of a test specimen. ie, surface in plane of movement.
c) Floculated kaolin from same sample as (b) but outside of failure zone
d) Random, powder pattern. Powder sprinkled on plate and X-rayed
Parallel
(003)
24 Å
(b)
Semi - parallel
(c)
Random
(001)
7.2Å
(hkℓ)
(002)
3.6 Å
(003)
2.4 Å
(d)
Random
8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40
DEGREES 2θ

FIGURE 6

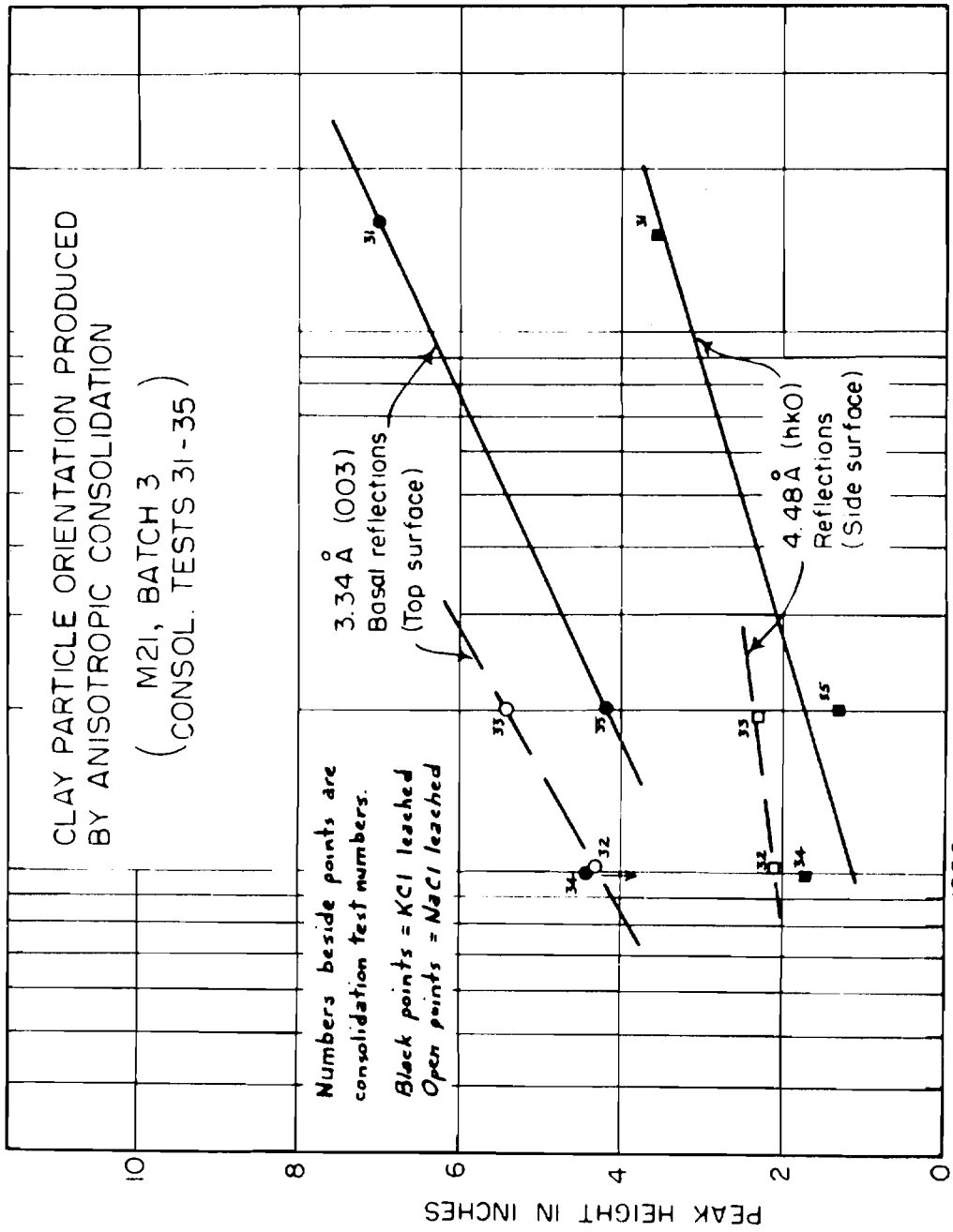
CLAY PARTICLE ORIENTATION PRODUCED BY ANISOTROPIC CONSOLIDATION
(M21, BATCH 3 CONSOL. TESTS 31-35)
Numbers beside points are consolidation test numbers.
Black points = KCl leached
Open points = NaCl leached
3.34 Å (003) Basal reflections (Top surface)
4.48 Å (hk0) Reflections (Side surface)
PEAK HEIGHT IN INCHES
MAXIMUM PREVIOUS CONSOLIDATION PRESSURE
1000
10,000
0
2
4
6
8
10

FIGURE 7

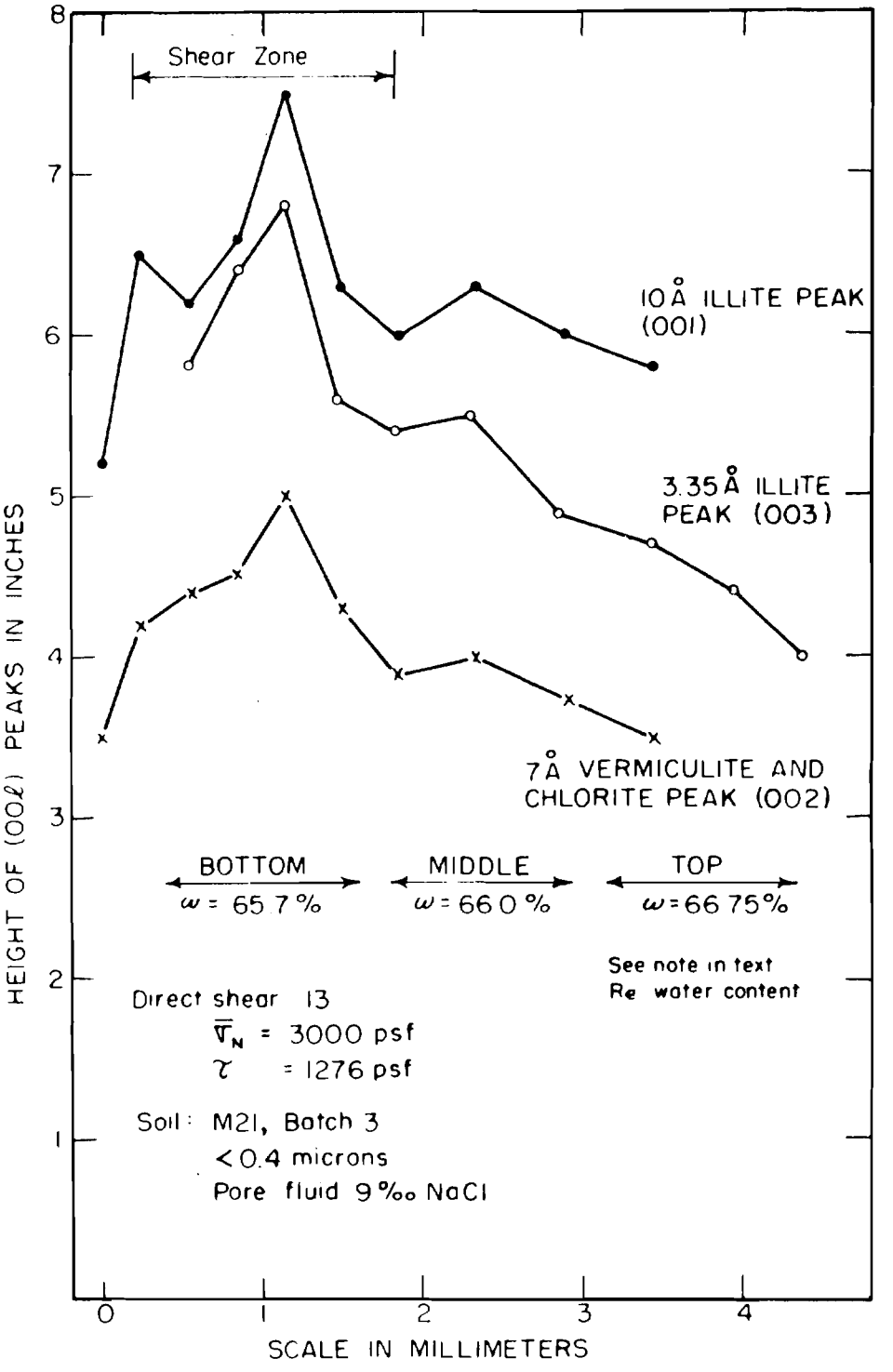

FIGURE 8 PARTICLE ORIENTATION WITHIN A SHEARED DRAINED DIRECT SHEAR SAMPLE (SAMPLE AIR DRIED BEFORE X-RAY)

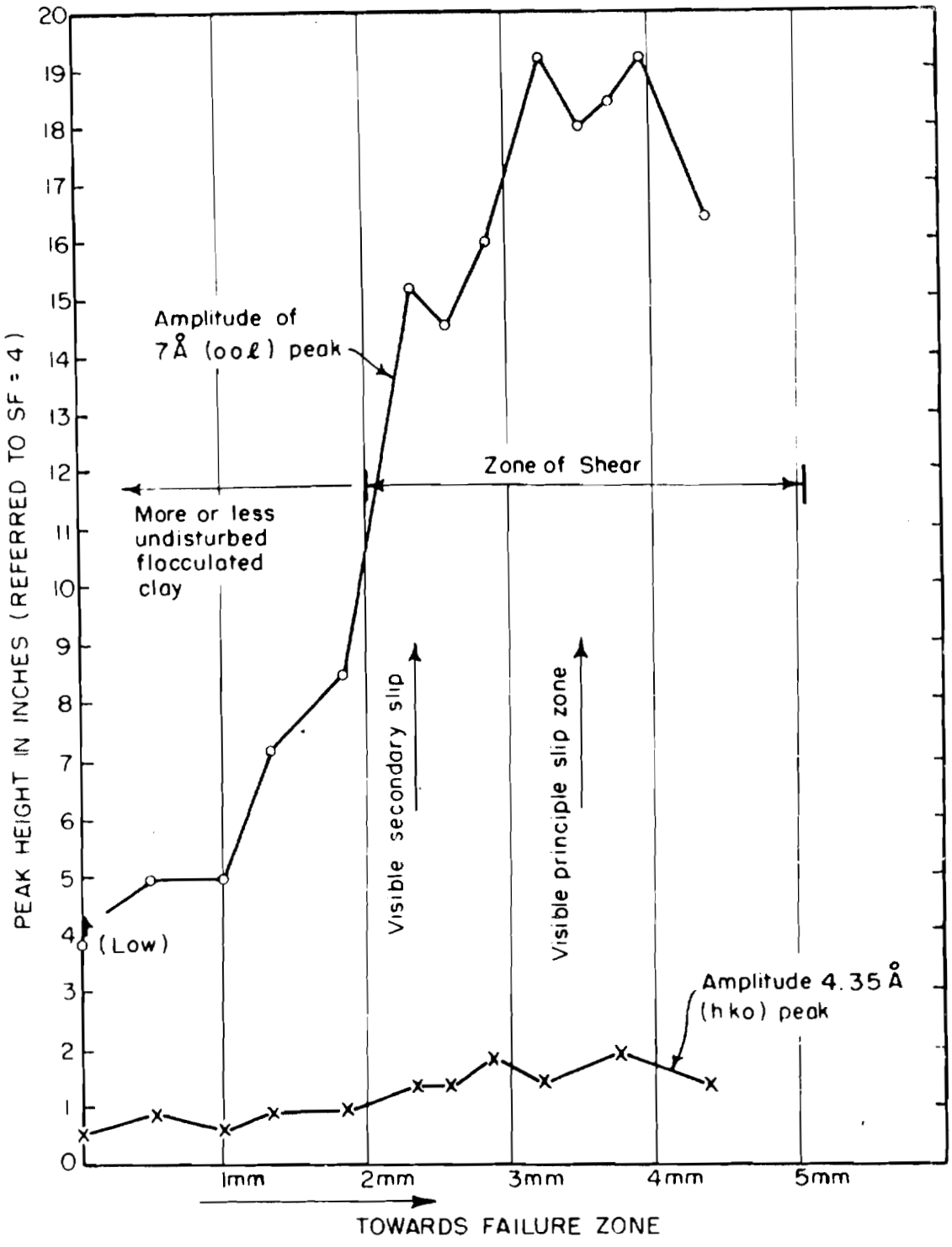

FIGURE 9 CHANGE IN PARTICLE ORIENTATION AS A RESULT OF SHEARING ACTION

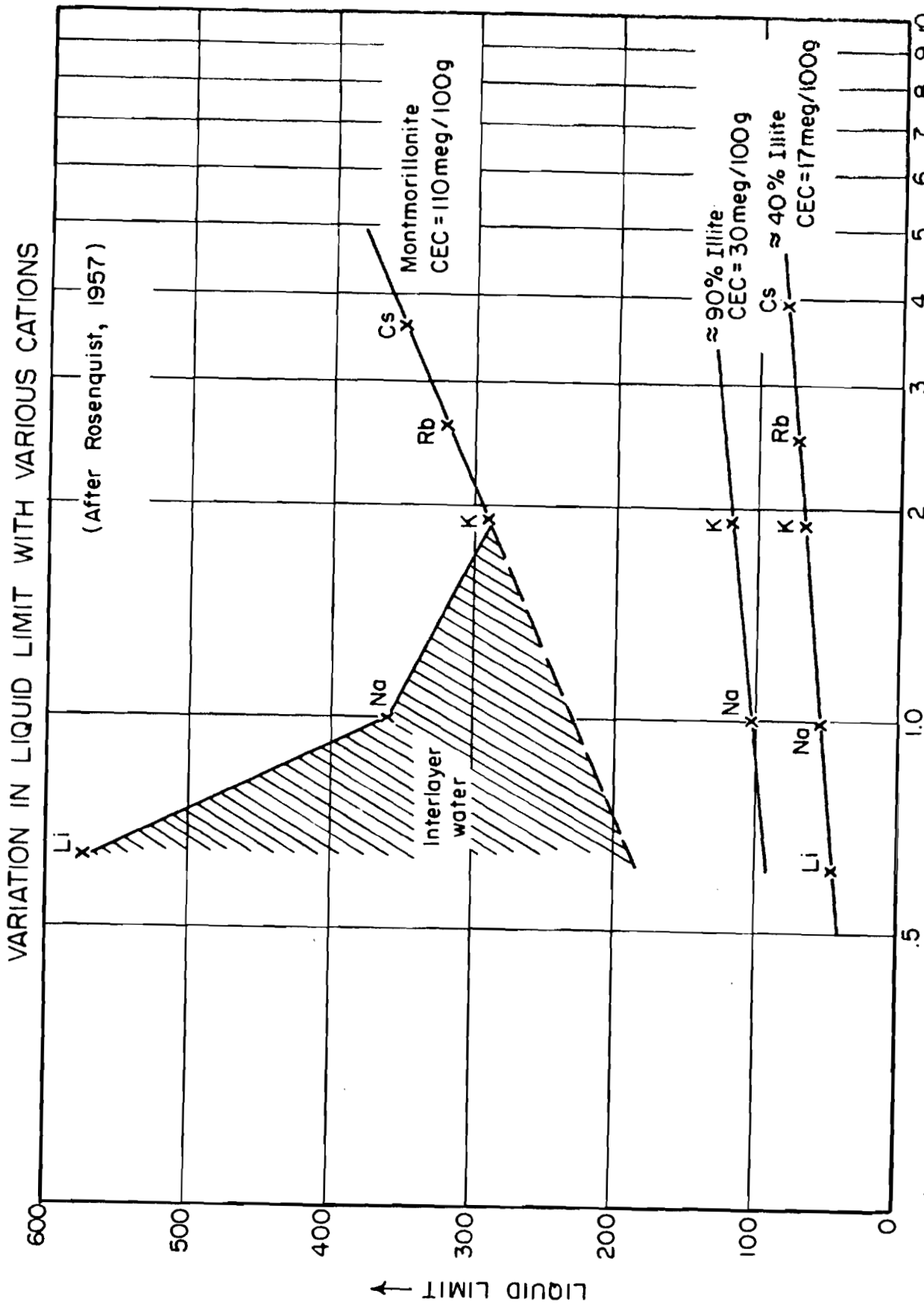
VARIATION IN LIQUID LIMIT WITH VARIOUS CATIONS
(After Rosenquist, 1957)
LIQUID LIMIT →
600
500
400
300
200
100
0
Montmorillonite
CEC = 110meg/100g
Li
Na
K
Rb
Cs
Interlayer water
≈ 90% Illite
CEC = 30meg/100g
≈ 40% Illite
CEC = 17meg/100g
.5
1.0
2
3
4
5
6
7
8
9
10
$\alpha = \frac{\epsilon - 1}{\epsilon + 2} R^3$ (Å^3)

FIGURE 10

MEASUREMENT OF SHEAR STRENGTH, PLASTICITY AND WATER RETENTION OF CLAYS RELATED TO INTERPARTICLE FORCES

by

B. P. Warkentin

SUMMARY

The action of forces between clay particles can explain the behaviour of soils in certain engineering applications. These forces are associated with the high surface area and electric charge of clays, and their action depends upon the composition and arrangement of the particles and upon the pore fluid. In naturally occurring soils, any quantitative manifestation of these forces is poorly understood. In this paper several examples are chosen from recent measurements on remoulded samples of well-defined clay minerals to show how engineering properties of clays can be interpreted on the basis of interparticle forces.

Montmorillonite clay can take up considerable amounts of water on swelling and retain its strength, whereas a kaolinite sample will fall apart. This difference is not simply related to forces of attraction; the strength of montmorillonite increases as the force of repulsion increases. Measured shear strengths of pure clays are given and interpreted on the basis of the forces required to move particles in the failure plane. For montmorillonite this movement is resisted by interparticle repulsion; for kaolinite the forces of attraction depending upon acidity and salt contentration are important.

The liquid limits of these clays are also interpreted from the dominant interparticle forces.

For clays which on drying shrink, with no increase in air-filled voids, water is retained by the same forces which cause swelling. The mechanism of water retention is of interest when the principle of effective stresses is applied where measured pore water pressures are

negative. Experimentally measured water retention curves for clays are compared with calculated swelling pressure to show the contribution of interparticle repulsion and attraction to water retention.

- * -

The behaviour of clay soils is determined by forces acting between clay particles. This action can be through the water layers intimately held at the particle surfaces through the diffuse layer of exchangeable cations, or in some cases direct particle contact. Forces between particles of clay size are studied within the discipline of physical chemistry; hence one speaks of these interparticle forces as physico-chemical forces. Interparticle forces in clays arise from the high surface area which actively absorbs water and organic molecules, from the electric charge which attracts and holds the exchangeable cations, and from inter-atomic forces.

Many recent studies have shown the influence of particle arrangement on engineering behaviour of clay soils (1) and methods have been devised for its measurement (2) although quantitative specification is difficult. This particle arrangement is the result of the action of interparticle forces. In sedimented clays, arrangement is the result of interparticle forces determined by the environment, e. g. concentration of salt in the water in which the clays were deposited determines whether they will have a flocculated or a dispersed particle arrangement.

Interparticle forces in clays are not completely understood. There is considerable qualitative information on how certain clays react to given empirical measurements, e. g. amount of swelling and the swelling pressure, and a qualitative and descriptive understanding of the nature of the forces which must be responsible. For some pure clays, the swelling pressure can be derived from the forces of repulsion (3).

Forces of attraction between particles, which are of more importance than forces of repulsion in engineering behaviour,

are more difficult to specify. Often they result from inorganic or organic "impurities" which bond one particle to another. In natural soils, which often contain a mixture of clay minerals as well as these "impurities" which react with the minerals, a quantitative specification of forces has rarely been achieved, and often even a qualitative understanding of their behaviour in terms of the controlling interparticle forces is lacking.

In view of these difficulties and uncertainties, several approaches to the problem of interparticle forces in engineering behaviour are possible but none of these alone is fully satisfactory or all-inclusive. The work in the Soil Mechanics Laboratory at McGill University commenced several years ago using prepared samples of pure clay minerals for which some information on interparticle forces was available. The engineering properties of these clays were measured and then related to interparticle forces to see in what way these forces were manifested in engineering behaviour. It has been found that such a relationship can often be established, but it is not simple or straightforward.

This work is part of a continuing program in the Department of Civil Engineering, under the direction of Prof. R. Yong, and the author. This paper will discuss some of the results obtained in the study to illustrate what is being learned from measurements on pure, especially prepared samples of clay minerals.

RESULTS AND DISCUSSION

Shear Strength

Measurements of resistance to direct, translatory shear were made on samples of clays prepared to have differences in interparticle forces. Shear strength was then related to the interparticle forces. Because of the uncertainty of interpreting direct shear values, no reliance was placed upon the precise values obtained. However, the differences in measured values with changes in interparticle forces are felt to be relevant. An account of the experimental methods and interpretation of results has been published (4).

It has been possible to delineate the interparticle forces responsible for shear strength in certain clays and to set up a

model, but the shear strength is not simply related to forces of attraction minus repulsion. Swelling and non-swelling clays act differently. Swelling clays - where repulsion can be the dominant interparticle force - retain strength even when wet. When non-swelling clays are wetted to the same water content, the samples fall apart. This difference is apparent in the marked tendency for non-swelling clays such as kaolinite to erode under moving water.

Figure 1 shows the shear strength values measured for sodium montmorillonite, a high-swelling clay, at different void ratios and at two salt concentrations. At the higher salt concentration, 1.0M NaCl, interparticle repulsion is lower, attraction is higher and shear strength is lower. In Figure 2, calcium montmorillonite with a lower swelling (lower net repulsion) and higher attraction, has the lower shear strength at the same void ratio or same average interparticle distance. In all of these measurements, shear strength increases as net repulsion increases. This increase in repulsion is accompanied by an increase in soil suction (see section on water retention), so that a greater strength would be predicted from effective stresses. However, this does not explain the existence of shear strength in a clay where the swelling shows that a net repulsion between particles exists.

During shear, particles in the failure plane become oriented parallel to each other, and any force resisting this orientation should contribute to shear strength of remoulded clays. Figure 3 suggests how interparticle repulsion can play this role for high-swelling clays. Formation of the failure plane requires particle movement, in some places the closer approach of one particle to another. Interparticle repulsion resists this movement, accounting qualitatively for measured strength values. It is difficult to apply this concept quantitatively because particle orientation is unknown and changes as void ratio changes, becoming more parallel as the void ratio decreases. Therefore, while net repulsion can be measured (as the swelling pressure), the amount of movement required can not be specified.

From the model it may be predicted that shear strength would be lower for a more parallel particle orientation. Support for this concept was obtained in the author's laboratory from measurement (in a modified way) of the liquid limit of a sample with parallel particle orientation and one with random orientation. At the same water content, the sample with parallel orientation required fewer blows to close the

groove. The test was not repeated, and it is not known whether all other factors were constant.

Figure 4 shows measured shear strength values for kaolinite, a low-swelling clay. Highest shear strength was measured for the flocculated clay, where attraction is the dominant interparticle force. The dispersed clay, with net repulsion, had lower strength at the same void ratio. Interparticle forces were still important in determining shear strength, as shown by the low values for the coarse fraction of the clay.

The forces of attraction between particles involved in flocculation are not sufficiently strong to contribute appreciably to shear strength in a direct way, i. e. in resisting separation of particles. Interparticle attraction could keep the particles in a certain orientation, contributing to strength by resisting the rearrangement required to form a failure plane. If the flocculated structure is visualized as a random arrangement of particles with face-to-face and edge-to-face contacts, forces of attraction would resist the rearrangement to parallel orientation. In the dispersed clay, repulsion keeps the particles in an arrangement which is more nearly parallel to begin with, requiring less force and less movement for rearrangement in the failure plane.

The measurements discussed above are representative of the results obtained in this study. Taking all the work into consideration, it is concluded that shear strength can be interpreted on the basis of interparticle forces for remoulded clays and that these forces lead to a specific particle arrangement and stability of that arrangement which accounts for resistance to shear. This depends upon both the force required to move particles and the amount of movement in the failure plane.

The results apply only to remoulded clays; for undisturbed soils, forces of attraction due to interparticle cementation are often dominant. However, for clays with sensitivity values close to unity, these considerations are pertinent.

Liquid Limit

The upper plastic limit, or liquid limit, is generally

interpreted as the water content just exceeding the amount of bound water. This small excess of free water allows the particles to move past each other. The thickness of the bound water layer is inferred from the water content at the liquid limit and varies with the clay and the exchangeable cation. Different exchangeable cations fall in a series of increasing bound water layer thickness, but it is found that the series does not remain the same for different clays. For swelling clays, monovalent exchangeable ions such as sodium give the higher liquid limit, whereas for non-swelling clays, divalent calcium or trivalent aluminum give the higher values. Measurements were, therefore, made of the liquid limit for samples for which interparticle forces could be specified to see whether plastic properties could be related to these forces.

For this interpretation, it has been found convenient to speak of an "interaction volume" defined as that volume within which particles or units of particles interfere with the movement of adjacent particles (5). The interparticle forces determining the interaction volume must then be specified.

Measured liquid limits for montmorillonite at different salt concentrations are shown in Figure 5. For sodium montmorillonite, interparticle repulsion determines the interaction volume. As repulsion is decreased by increasing the salt concentration, interaction volume and liquid limit decrease. Repulsion keeps the particles fixed at a certain distance from each other and when this repulsion is decreased, the particles have a greater freedom for movement. Substituting a divalent ion, calcium, also decreases repulsion and hence the liquid limit. But increasing salt concentration has been shown to lead to only a small decrease in repulsion and it decreases the liquid limit by only a small amount.

The dominant force in kaolinite is again interparticle attraction keeping the particles in a fixed orientation and defining the interaction volume. The dispersed kaolinite, with maximum repulsion, has the lowest liquid limit at low salt concentration in Figure 6. The particles are in more nearly parallel orientation and since repulsion is low for the low-swelling clay, the interaction volume is small. The clay with edge-to-face flocculation due to positive charge on the edges and negative charge on the planar surfaces, has the highest interaction volume.

With increasing salt concentration, all samples change to a face-to-face structure due to salt flocculation with an intermediate interaction volume.

The effect of exchangeable cations on liquid limit of clays can be understood from these results. For high-swelling clays, polyvalent cations decrease repulsion and hence the liquid limit; for low-swelling clays they increase flocculation and increase the interaction volume and liquid limit. The dominant interparticle forces differ.

Water Retention

Interparticle forces are also involved in retention of water by clay soils. If the shrinkage on drying equals the volume of water lost, the clay remains water-saturated. However, the measured pore-water pressure would be negative and if the soil were placed in contact with free water it would take up water and increase in volume. Since effective stresses are used in the analysis of strength of soils at these water contents, it is of interest to examine water retention more closely.

The force that a dry soil can exert in the absorption of water, measured as the restraining force that must be exerted on the water to prevent movement, is called soil suction. Soil suction decreases as the water content increases. This is not a water tension as the water is not necessarily under tension or negative pressure equal to the value of the soil suction. Pore water pressure measured through a fine porous stone as a negative head of water may be equal to soil suction but not necessarily equal to the pressure on the soil water. To prevent confusion, it is best to think in terms of thermodynamics of a free energy difference between the water in the soil and free water.

Water retention forces in sands differ from those in clays. When a saturated sand loses water, air enters the voids. The pressure difference across the curved air-water interface gives rise to capillary water retention and we speak of capillary forces. The force of retention, or capillary rise, depends upon the size of voids.

In a clay soil which is water-saturated, there are no air-water interfaces and change in water content is accompanied by a change in volume. The force required to remove an increment of water is that required to decrease the volume by that amount (assuming no

direct interparticle interference). This force is equal to the swelling pressure (8). Therefore, making allowance for units and dimensions, the swelling pressure equals the force of water retention at that water content. One mechanism for the swelling pressure is the excess ion concentration between clay particles resulting in an osmotic gradient which causes water to move between the particles. Swelling pressure is the pressure necessary to prevent this water movement (6).

The expected equivalence between swelling and water retention has been checked for several clays (7). The results shown in Figure 7 for one clay are representative. Measured swelling pressures were not available for this clay; the calculated values were obtained from equations giving the ion concentrations around clay particles (7). These calculations give only the repulsion and are in error by the amount of forces of attraction between particles.

Measured suction values agree with calculated swelling pressures for the slurried sample down to pF 2.5 or about 300 cm. of water suction. The pF is the logarithm of the measured soil suction in cm. of water. Below that, forces of attraction apparently prevent swelling and uptake of as much water as predicted from repulsion. These interparticle attraction forces are important in soil structure and are being studied further. The swelling pressure overestimates the water content of the undisturbed sample at all suction values. Forces of attraction apparently operate over the entire range. These comparisons have indicated that interparticle forces account for water retention over part of the suction range for the clays studied.

Particle arrangement also influences water retention by clays. High-swelling clays have the highest water content for parallel particle orientation but low-swelling clays have the highest water content for random arrangement where extra water beyond that involved in swelling is trapped between irregularly arranged particles or aggregates of particles. The high natural water content of Leda clay, up to 70%, is due to the random particle arrangement. If the arrangement is disturbed and the sample dried, the maximum water content decreases to about 35%.

Acknowledgment

This work was carried out under the research grants from the National Research Council whose assistance is gratefully acknowledged.

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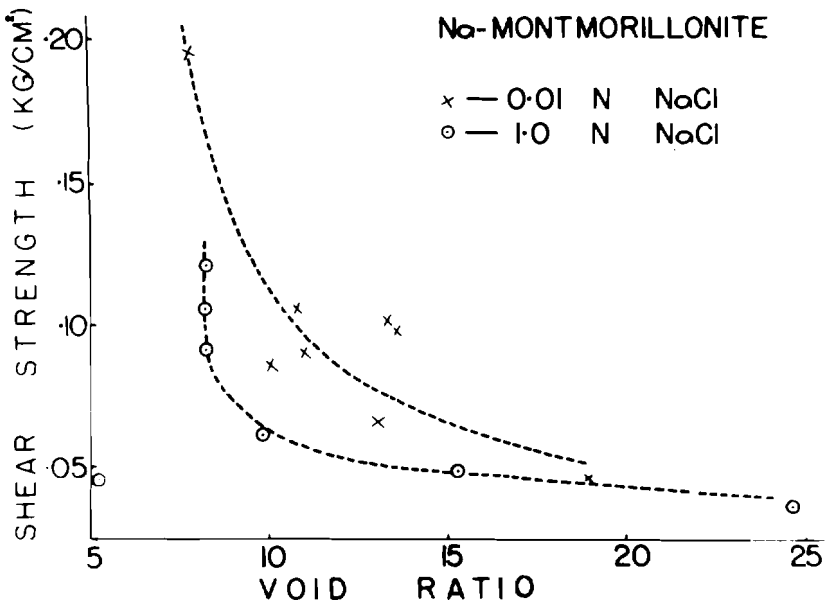

Fig. 1 Measured shear strength of sodium montmorillonite at different salt concentrations in the pore water.

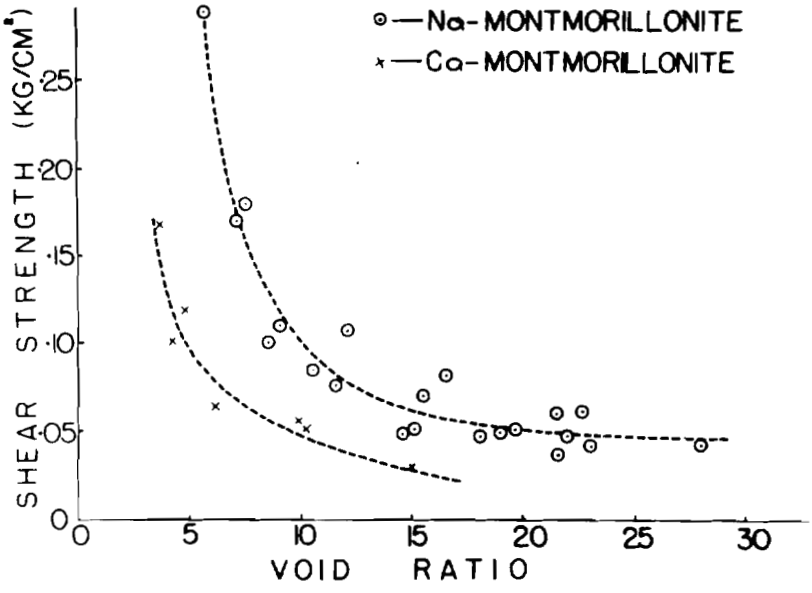

Fig. 2 Measured shear strength of sodium and calcium montmorillonite.

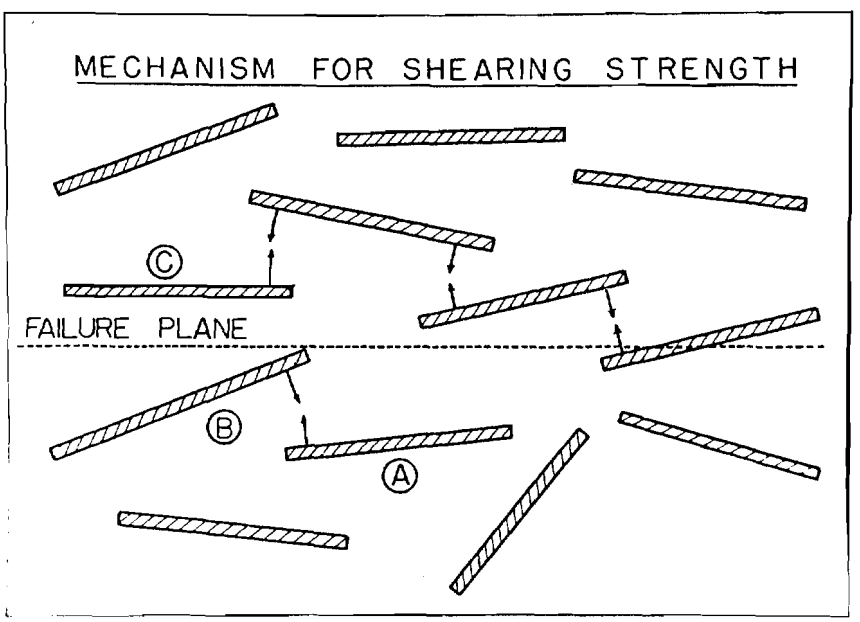

Fig. 3 Schematic diagram for model of interacting clay plates with interparticle repulsion resisting particle rearrangement during development of failure plane.

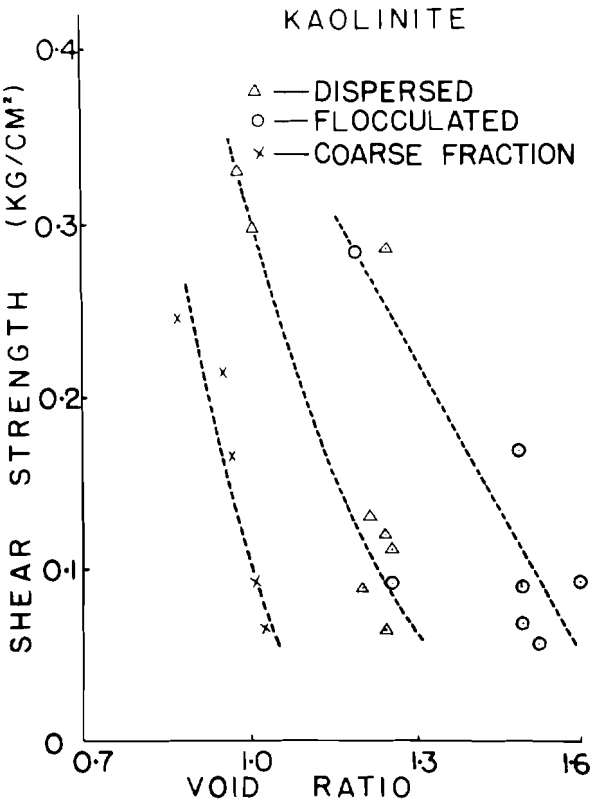

Fig. 4 Measured shear strength of flocculated and dispersed kaolinite and of coarse kaolinite.

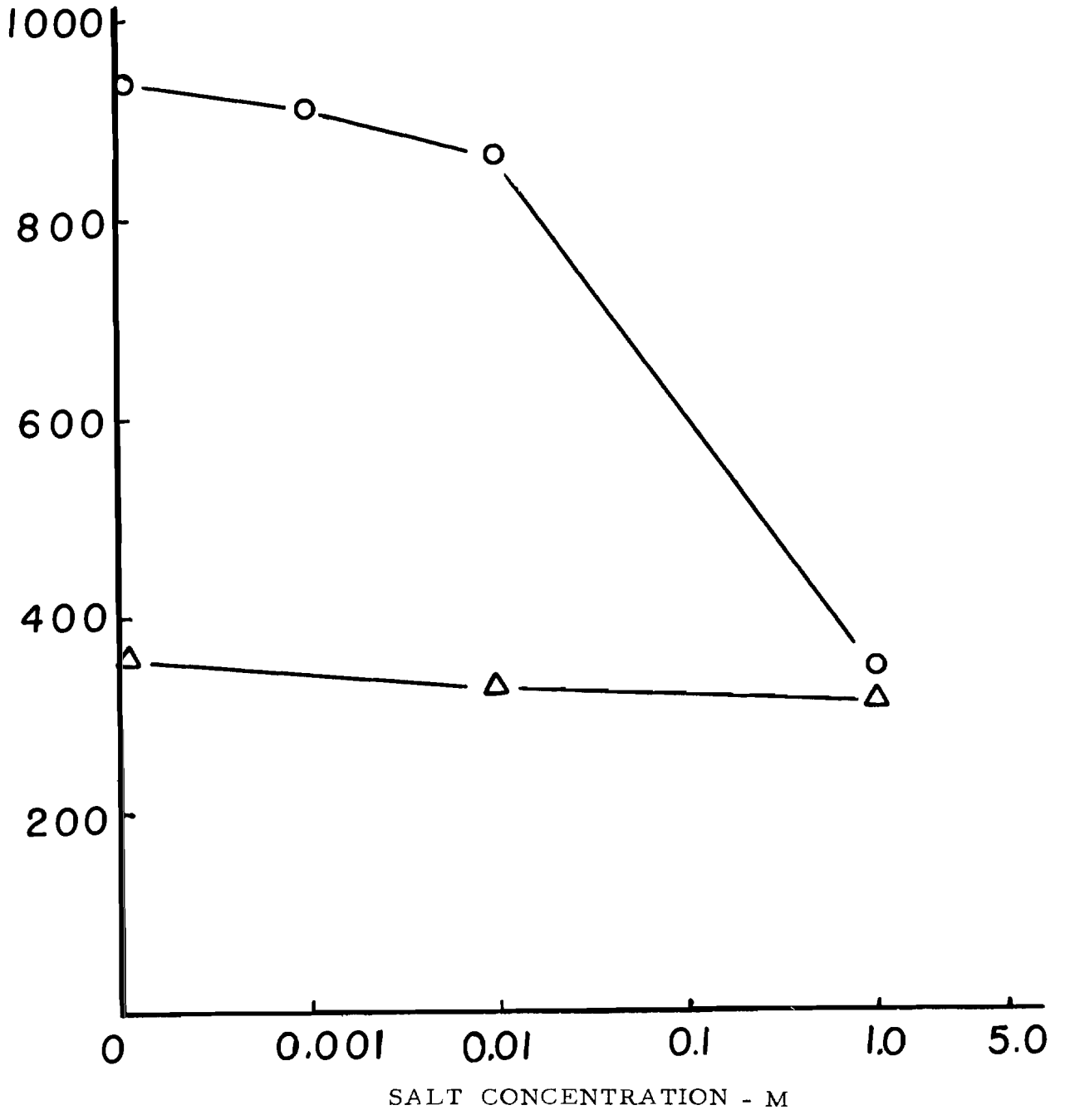

Fig. 5 Measured water content at the liquid limit for montmorillonite clays (0-Sodium Montmorillonite + NaCl; X-Calcium Montmorillonite + $CaCl_2$).

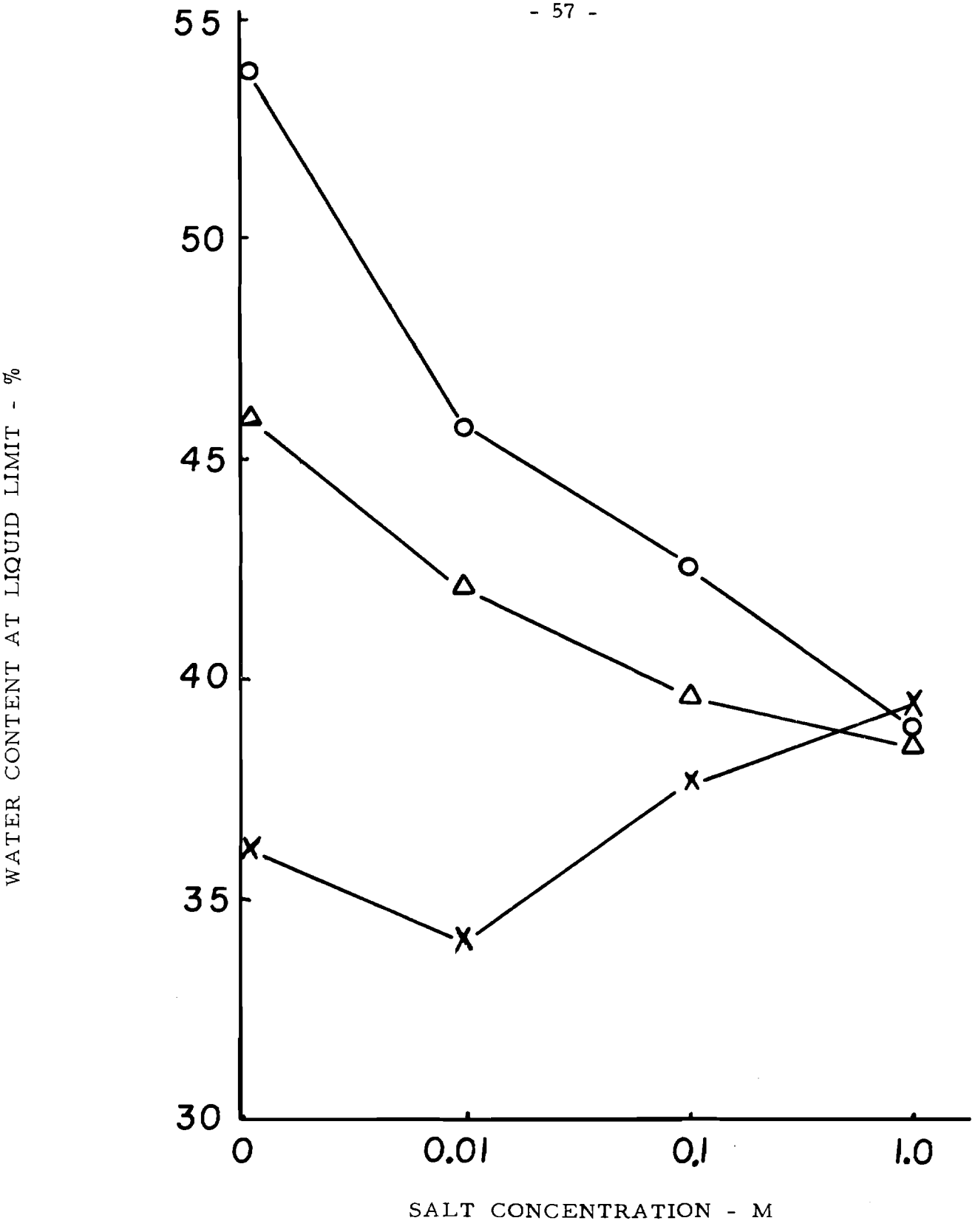

Fig. 6 Liquid limit of kaolinite samples with different interparticle forces (O-Flocculated, edge-to-face at pH 4, with addition of $CaCl_2$; Δ -Flocculated at pH 6, with addition of $CaCl_2$; X-Dispersed at pH 10, with addition of NaCl).

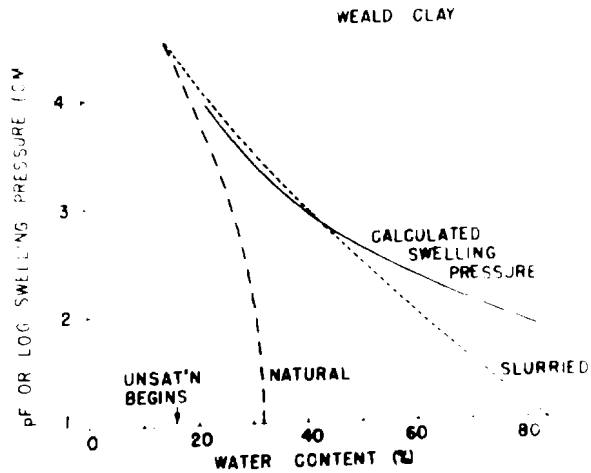

Fig. 7 Measured water retention and calculated swelling pressure for Weald clay. (Water retention values from Croney et al, 1953).

HEAVE OF SPILLWAY STRUCTURES ON CLAY SHALES

by

R. Peterson and N. Peters

SUMMARY*

Spillways constructed on clay shale in Western Canada have heaved considerably. This paper briefly describes the clay shales of the area and gives typical laboratory swelling test results. It has not yet been possible to predict total heave and rate of heave from laboratory tests. Observational programs to check the performance of existing structures as well as special tests were undertaken to provide information required for design. Typical heave observations at spillways showing the effect of seepage, frost action, and hold-down piles are included.

* Summary prepared by authors.

Note:
Paper published in full in the Canadian Geotechnical Journal, Vol. I, No. 1, September 1963.

VOLUME CHANGES IN UNDISTURBED CLAY PROFILES IN WESTERN CANADA

by

J. J. Hamilton

SUMMARY*

As part of an overall study of the performance of building foundations in highly plastic soil areas of Western Canada, the Division of Building Research has been measuring ground movements and changes in soil moisture conditions in grass-covered, undisturbed soil profiles under climatic conditions ranging from sub-humid to semi-arid. Results of measurements begun in 1951 in Winnipeg, Manitoba, and more recently those in Regina, Eston and Tisdale, Saskatchewan, are reported. Results of a theoretical soil moisture depletion calculation, based on Thornthwaite's potential evapotranspiration concept, are presented. It is proposed as a more rational way of measuring the vegetation-climate factor in humid to sub-humid climates than simply comparing air temperature and precipitation with long-term averages. Empirical relationships are suggested between calculated soil moisture depletion, the depth of free water table, and the depth of frost penetration under similar thermal conditions but different soil moisture conditions. In situ shrinkage of undisturbed, unsaturated soils at temperatures well below 32°F has been observed and is attributed to thermal air-void volume change.

* Summary prepared by author.

Note:
Paper published in full in the Canadian Geotechnical Journal, Vol. I, No. 1, September 1963.

PRACTICAL EXPERIENCE WITH HIGHLY SWELLING SOIL TYPES

by

A. O. Dyregrov and R. M. Hardy

SUMMARY*

The problem of the effect of swelling soil on buildings with shallow foundations, on the stability of slopes, and on roads and runways, is of considerable importance in areas of recent glacial lake deposits in Western Canada. It is thought that the physico-chemical nature of the swelling pressure tends to reduce the effective stress but is not reflected in the pore pressure in the water phase of the soil. The suggestion is made that the effective stress be calculated by subtracting the pore pressure from the total stress in the usual way and in addition P_s, the swelling pressure, should also be subtracted. From a practical point of view, this term should be calculated from free-swell consolidation tests rather than constant volume test.

*Summary prepared by E. Penner.

Note:
Paper submitted for publication to the EIC.

PHYSICOCHEMICAL PHENOMENA IN SOIL MATERIALS

by

S. Pawluk

SUMMARY*

Particle size distribution of soil separates falls essentially into three basic groups, sands, silts, and clays, the size distribution limits of which remain controversial among the various scientific groups interested in their study. Soil scientists have for a long time accepted the division between clay and coarser fractions as 2 microns, since, in general, it is the fraction less than 2 microns in size that contains clay minerals which exhibit physico-chemical manifestations related to colloidal behaviour.

Among the various physico-chemical features observable, the most significant are those related to ion exchange phenomenon and water adsorption. It is these factors which are responsible for many of the anomalies observed when studying soil materials from a purely physical point of view.

Ion adsorption and exchange phenomena characteristic of colloidal clay minerals result from the fundamental nature of their structural units in which isomorphous substitution, broken bonds on crystal edges, and exposed hydroxyl groups play an essential part. The resulting effect of these structural characteristics is the presence of a net negative charge at the colloidal surfaces capable of preferentially attracting positively charged genenions as well as polar solvents, such as water, when placed in aqueous electrolyte solutions.

*Summary prepared by author.

Note:
Paper submitted for publication to the EIC.

SURFICIAL DEPOSITS OF ALBERTA

by

C. P. Gravenor and L. A. Bayrock

SUMMARY*

The surficial deposits of central and southern Alberta have been under study intermittently for some seventy-five years. Most of the early work was confined to the examination of sections along the major river courses and to broad reconnaissance studies. After the Second World War detailed mapping of the surficial deposits of Alberta was started by the Geological Survey of Canada and the Research Council of Alberta. To date, approximately 40,000 square miles of central and southern Alberta have been mapped in detail. In addition, the Research Council of Alberta has mapped about 60,000 square miles of northern Alberta over the past four years by means of helicopter surveys. Thus, roughly forty per cent of the surface deposits of the Province have been mapped. The helicopter program to map the area north of fifty-seven degrees latitude will be completed this year and it is anticipated that about one-half of the Province will have been mapped by 1963.

Detailed soil maps covering parts of the Peace River Country, central and southern Alberta, and reconnaissance maps of northern Alberta have been prepared by the Alberta Soil Survey. Although in some cases the nomenclature used by soil scientists for the parent materials of the soils is not identical with that used by geologists, these maps are nevertheless extremely useful to the geologist and provide an excellent background for detailed geological study of surficial materials.

This paper is designed to provide a generalized picture of the nature and distribution of the surficial deposits of Alberta. Particular attention will be given to a description of buried channels which mark the location of the drainage system which existed in Alberta prior to glaciation.

*Summary prepared by author.

Note:
Paper submitted for publication to the Canadian Geotechnical Journal.

ENGINEERING ASPECTS OF THE GREAT SLAVE RAILWAY

by

J. L. Charles, V. R. Cox and F. L. Peckover

SUMMARY*

The 430 mile railway line running from Peace River to Hay River with a branch line to Pine Point Mines, 50 miles east of Hay River, is scheduled for completion by December 1965. The geology of the route and the nature of the terrain is described as well as how this knowledge was utilized in circumventing difficult engineering problems. The route is located in an area of extensive muskeg traverse the southern fringe of the permafrost region but no large areas of permafrost have been encountered.

*Summary prepared by E. Penner.

Note:
Paper submitted for publication to the EIC.

PRELIMINARY SOIL MECHANICS ASPECTS OF THE RED RIVER FLOODWAY

by

J. Mishtak

SUMMARY*

Winnipeg, situated at the confluence of the Red and Assiniboine Rivers, has been plagued by floods of large magnitude since 1826. Subsequent large floods occurred in 1852, 1861, 1916, 1948 and 1950. The flood of 1950 caused such severe damage as to stimulate positive action towards alleviation of the flood problem. As a result, the Dominion Government set up the Red River Basin Investigation, whose purpose was to investigate various possibilities of alleviating flooding. Subsequently, the Royal Commission on Flood Cost Benefit was set up to select the most economically feasible combination of schemes submitted by the Red River Basin Investigation. The result was the selection of a diversion channel bypassing the Greater Winnipeg area in a combination with several other projects. Studies by the Water Control and Conservation Branch of the Province of Manitoba resulted in the relocations of the diversion channel from the alternative recommended by the Red River Basin Investigation. This diversion channel is known as the Red River Floodway.

The Red River Floodway will be about 30 miles long, with a top width of 1,000 ft and an average depth of 30 ft. The quantity of excavations is some 100 million cubic yards. Its design capacity of flow is 60,000 cfs.

The soil to be excavated is a highly plastic clay of glacial Lake Agassiz underlain by glacial till. The main design problem of the channel was the selection of slopes which would be stable almost indefinitely. Other problems involved the effect of rebound on excavated grades, erosion, embankment composition, methods of excavation, groundwater seepage during excavation, depletion of local groundwater supplies and foundation of structures.

*Summary prepared by E. Penner.

Note:
Paper submitted for publication to the Canadian Geotechnical Journal.

THE MICA CREEK PROJECT - FOUNDATIONS AND MATERIAL INVESTIGATION

by

W.P. Harland

SUMMARY*

This project is a part of the proposed power development of the Columbia River and is located between Golden and Revelstoke where the river has its steepest gradient of 1000 ft per 200 miles. In addition to describing the selection of the site, the geology of the area is discussed in some detail based on the foundation investigations. To date, about 4 miles of diamond drilling in bedrock and overburden have been completed. Two major soil types fill the main bedrock channel; dense glacial till and a deposit of sand and gravel with varying proportions of boulders. It is believed that a high fill-type dam at Mica Creek has been shown to be feasible based on foundation conditions and the availability of suitable fill and other material required.

*Summary prepared by E. Penner.

Note:
Paper submitted for publication to the EIC.

- * -

FOUNDATION TREATMENT AND CONSTRUCTION OF SIXTEEN MILES OF DYKE AT MANITOBA HYDRO'S GRAND RAPIDS PROJECT

by

J.R. Rettie, A. Koropatnick and W.S. Isom

SUMMARY

The project, now in its second year of construction, is located at the mouth of the Saskatchewan River where it flows into Lake Winnipeg, 250 miles north of the City of Winnipeg. The area has been glaciated, is underlain with fragmental and massive limestone, dolomite deposits of the Palaeozoic age and is covered with numerous sink holes. The extensive grouting and foundation treatment required to prevent leaking from the reservoir is described in detail, as is the design and construction of the 16 miles of dyke necessary to enclose the forebay. The outstanding feature of the project is the large grouting operation made necessary by the foundation condition of the site.

*Summary prepared by E. Penner.

Note:
Paper submitted for publication to the EIC.

- * -

APPENDIX "A"

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